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A SERIES OF TEXTBOOKS FOR PERSONS ENGAGED IN ENGINEERING PROFESSIONS, TRADES, AND VOCATIONAL OCCUPATIONS OR FOR THOSE WHO DESIRE INFORMATION CONCERNING THEM. FULLY ILLUSTRATED

STONE AND BRICK
CEMENTING MATERIALS AND MORTAR
STONE AND BRICK MASONRY
PLAIN CONCRETE
REINFORCED CONCRETE
FOUNDATIONS
RETAINING WALLS
CULVERTS
TUNNELS
DAMS
INTRODUCTION TO CONSTRUCTION DRAWING

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PREFACE

The volumes of the International Library of Technology are made up of Instruction Papers, or Sections, comprising the various courses of instruction for students of the International Correspondence Schools. The original manuscripts are prepared by persons thoroughly qualified both technically and by experience to write with authority, and in many cases they are regularly employed elsewhere in practical work as experts. The manuscripts are then carefully edited to make them suitable for correspondence instruction. The Instruction Papers are written clearly and in the simplest language possible, so as to make them readily understood by all students. Necessary technical expressions are clearly explained when introduced.

The great majority of our students wish to prepare themselves for advancement in their vocations or to qualify for more congenial occupations. Usually they are employed and able to devote only a few hours a day to study. Therefore every effort must be made to give them practical and accurate information in clear and concise form and to make this information include all of the essentials but none of the non-essentials. To make the text clear, illustrations are used freely. These illustrations are especially made by our own Illustrating Department in order to adapt them fully to the requirements of the text.

In the table of contents that immediately follows are given the titles of the Sections included in this volume, and under each title are listed the main topics discussed. At the end of the volume will be found a complete index, so that any subject treated can be quickly found.

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CONTENTS

STONE AND BRICK

STONE

INTRODUCTION

1. The materials employed in the construction of masonry are *stone*, *brick*, *terra cotta*, and the *cementing materials*—cement, lime, and sand—used in the manufacture of mortars. In order that enduring structures may be erected, it is necessary to have a knowledge of: (1) the source of the materials; (2) their properties; (3) the methods of preparing them; and (4) the manner of placing them in the structure.

2. Rocks of many different kinds are found in the earth's crust, but only a few are suitable for structural purposes, owing to the difference in quality arising from the difference either in chemical composition or in physical structure. These differences frequently fit the various stones for special purposes.

The properties that determine the fitness of a stone for constructive purposes, and regarding which accurate knowledge is most essential, are:

1. *Durability*, or the ability of the stone to resist, for a considerable length of time, the disintegrating action of the several physical and chemical agents to which it will be exposed. Durability is affected by both the chemical composition and the physical structure of the stone, as well as by the character and position of its exposed surfaces.

2. *Strength*, or the ability of the stone to resist rupture under the stresses to which it is subjected.

3. *Hardness*, or the capability of the stone to resist abrasion. This is frequently an important factor, especially if the stone is to be subjected to frictional wear, as in steps and pavements; also, when the stone is to be used for quoins (corners) where it is necessary to preserve a sharp angle or arris.

4. *Weight*.—The weight of a stone has occasionally to be considered, as in massive work, where it is advisable to use heavy stones to resist the force of the sea or of the wind; also, in the construction of retaining walls, and of all structures whose stability depends, to a great extent, on their weight. In constructing arches, a stone of light weight is generally the most desirable.

5. *Appearance and Color*.—Esthetics requires that the appearance should be pleasing and attractive, and the color durable.

6. *Cost*.—Economic considerations require that the cost of procuring and preparing the stone should be low.

The extent to which a stone possesses some of these various properties can be approximately ascertained by tests and experiments, which will be considered under their proper heads.

CLASSIFICATION AND GENERAL CHARACTERISTICS OF ROCKS

3. Definitions.—The term **rock** is employed to denote the masses of mineral matter composing the earth's crust. In engineering construction work, the word **stone** is applied indiscriminately to all classes of hard rocks.

4. Bases of Classification.—The rocks from which building stone is obtained are described and classified according to: (1) their origin and position in the earth's crust, (2) their physical structure and appearance, and (3) their chemical composition.

GEOLOGICAL CLASSIFICATION

5. The strata that compose the earth's crust are classified by geologists into: (1) four great divisions, according to the animals and plants, extinct or living, that they contain; and (2) three great classes, according to the manner of their origin. The first classification, while of much importance to the student of geological history, has little interest for the constructor. The second classification, relating to the formation of the rock masses, has some value in determining the probable durability of a stone.

6. The three classes of the second classification are designated as follows:

1. The **igneous** (produced by fire), or **azoic** (devoid of life) **rocks.** Rocks of this class owe their formation to the solidification of molten minerals, and are represented by the lavas and basalts.

2. The **sedimentary,** or **aqueous, rocks,** which have been formed by (*a*) the chemical precipitation of mineral matter from water, as exemplified by the sandstones and limestones; (*b*) the action of animals and plants, as shown by coral; or (*c*) the mechanical destruction, and subsequent deposition, usually by water, of other rocks, as exhibited by the sands and clays.

3. The **metamorphic rocks,** formed by the transformation, or change in structure, of both the igneous and the sedimentary rocks, through the influence of heat or chemical action. To this class belong the granites, the marbles, gneiss, and the slate rocks.

PHYSICAL CLASSIFICATION

7. According to their physical structure, rocks are divided into two principal classes; namely: *stratified* and *unstratified rocks*.

8. **Stratified Rocks** are the sedimentary rocks of the geological classification. They are composed of grains bound together by a cementing medium, and usually consist

of a series of parallel layers indicating their deposition from water. The layers were originally horizontal, but in most cases they are found more or less inclined and curved through the action of disturbing forces. The strength and durability of these stones depend on the nature of the cementing material: when it is silicious, the stones are durable; but when it is alumina mixed with iron, the stones are likely to be speedily disintegrated.

9. The **unstratified rocks** belong to the igneous and metamorphic classes. They are, for the most part, composed of an aggregation of crystalline grains connected together without the interposition of a cementing material. These grains separate from each other when the rock decays or when it is struck violent blows. The line along which the rock separates or splits is called the **rift**, or **line of cleavage**; and in the quarrying of this class of rocks, the work is much facilitated by a knowledge of that line.

10. When the adhesion of the grains of an unstratified rock is weak, so that the rock can be easily broken, the rock is said to be **friable** or **loose textured**; when the grains adhere closely and strongly resist fracture, the rock is described as **hard** or of a **compact texture**. The character of the aggregation controls the working qualities of the stone. If the grains are hard but loosely coherent, the stone works easily; if the grains are soft and the cohesion is strong, the stone can be worked with difficulty.

When the grains measure nearly the same in length, breadth, and thickness, the structure of the rock is said to be **granular**; when they are easily discernible by the eye, **coarse granular**; when they are invisible to the unaided eye, **compact granular**; when they are thin and flat, **slaty**; when both classes of grains are present, **granular slaty**; when they are not crystalline, and the cohesive strength is low, **porous granular**; when they are crystalline, and the strength of cohesion is high, **compact crystalline.**

11. When one of the constituent materials forms a matrix in which the other constituents are embedded in the

form of fine grains or crystals, the structure is termed **porphyritic.** The porphyritic structure is so noticeable that all rocks possessing its characteristics in a marked degree are commonly termed **porphyries,** without regard to the mineral composition.

12. The term **concretionary** is applied to a structure made up of rounded particles, disposed in concentric layers, like the coating of an onion. When the concretions are small, like the roe of a fish, the structure is called **oolitic;** when large, like a pea, **pisolitic.** The term **conglomerate** is applied when the structure consists of fragments of one material, embedded in a mass of another.

13. The character of the structure of an unstratified stone is indicated by the manner in which the stone breaks or fractures. When the fracture is even and the surface of division is a plane free from inequalities, a crystalline structure is indicated, and the fracture is said to be **straight** or **right.** When the fracture is uneven, presenting a rough surface with sharp projections, it indicates a granular structure, and the fracture is called **hackly** or **splintery.** When the fracture resembles a shell, with convex and concave surfaces, it indicates a hard, compact structure; this type of fracture is known as the **conchoidal,** and by the workmen is called **plucky:** stones breaking in this manner are usually difficult to work. When the fracture has a rough, dull surface, the structure is called **earthy;** such a fracture indicates softness and brittleness.

CHEMICAL CLASSIFICATION

14. Chemically, stones are divided into three great classes; namely: **silicious stones,** in which silica is the predominant substance; **argillaceous stones,** in which alumina is the predominant substance; and **calcareous stones,** in which calcium is the predominant substance. Some or all of these substances are always found mixed or combined in different proportions.

15. Silica (Oxide of Silicon).—Silica, when pure, is white; it is incombustible, infusible, nearly insoluble in water, and not acted on by common acids. It is found in almost all stones whose structure is granular crystalline. Such stones, when of compact texture (see Art. **10**), are extremely durable, but difficult to work. Decay in silicious rocks generally arises from the decomposition of some softer mineral in the cementing medium; or, when they are of a porous texture, from the freezing of water in the pores. Fire causes them to scale and break off in fragments.

Rocks in which pure silica is the essential constituent are called **quartz rocks**, or simply **quartz**. They are usually easily recognized by their colorless appearance, irregular vitreous or glass-like fracture, hardness, and entire insolubility in acids. The hardness of quartz is such that it cannot be scratched with a knife. This mineral is, however, brittle; consequently, quartz rock works more easily than rocks from which quartz is absent.

16. Alumina (Oxide of Aluminum, Pure Clay). The stones that derive their peculiar properties from alumina are called **aluminous stones**; but, as they generally contain a mixture of silica also, they are described under the general name of argillaceous stones. Alumina, when pure, is white; it is incombustible and insoluble in water, but is soluble in acids and alkaline solutions. Alumina has a considerable attraction for the metallic oxides and other earths, and thus forms many complex combinations, known as *feldspars*, *micas*, *hornblendes*, and *pyroxenes*.

17. The combinations of alumina and silica with other elements, such as potash, soda, and lime, are termed **feldspars.** Several varieties are distinguished, each of which differs slightly from the others in composition and form. The most commonly occurring are *orthoclase* (potash), *anorthite* (lime), *albite* (soda), *labradorite* (lime and soda), *oligoclase* (soda and lime).

18. The feldspars are variable, both in color and quality; they are light in weight and almost as hard as quartz.

While quartz may be distinguished by its lack of cleavage, feldspar has distinct cleavage planes. It is always crystalline, though good crystals are not common. On exposure to the weather, it begins to change; under the action of water containing traces of carbonic acid, it first loses its lime, if a lime feldspar, by a combination of the lime with this acid; next, its alkalies are carried off as carbonates; if the supply of acid continues, the change ends in forming *kaolin* (fireclay), or some other hydrous silicate.

19. Combinations of silicate of alumina with alkaline metals, as potassium, lithium, magnesium, etc., are described by the general name **mica.** The micas vary in color from white, called **muscovite,** to black, called **biotite.** Both kinds occur in small shining scales, and are alike in having perfect cleavage, affording very thin, tough laminæ or sheets. They decay rapidly, forming soluble and insoluble products, the soluble parts passing off in water, the insoluble parts usually remaining as clayey deposits. On exposure to the action of the atmosphere, the black mica, owing to its large percentage of iron, becomes coated with a film of rust (oxide of iron), and then rapidly disintegrates. The white mica possesses greater endurance and remains intact for a long period of time.

20. The **hornblende,** or **amphibole,** group covers combinations of magnesia, lime, and iron, with alumina and with other minerals.

Hornblende can be recognized by its dark-green or almost black color, and by the tenacity and compactness of its crystals; it is distinguished from biotite mica in that it is not easily separable into thin leaves. Hornblende is a constituent of much importance on account of its toughness, strength, and durability; it is easy to polish, and its presence is considered more favorable to durability than mica.

21. The **pyroxene** group includes combinations of alumina with lime, magnesia, iron, and manganese, and sometimes soda, potash, and zinc; it includes also combinations of those minerals from which alumina is absent.

Thus, as with the amphiboles, two principal varieties are recognized—the aluminous and the non-aluminous. Pyroxene occurs as a compact, tough, yellowish-green, or black mineral, and cannot usually be distinguished by the unaided eye from hornblende, for their composition is nearly identical, and they are frequently found associated together in the same rock. Pyroxene is a common constituent of crystalline limestone and dolomite, and of serpentine and the volcanic rocks, and occurs also, but less abundantly, in granitic rocks and metamorphic schists. The pyroxene of limestone is mostly the white and light-green or the gray variety; that of most other metamorphic rocks is sometimes white or colorless, but usually green of different shades, and occasionally black.

22. Calcium and Lime.—Calcium forms the main constituent of calcareous rocks, which are also called **limestone rocks,** or simply **limestone.** It is the essential constituent of marble, chalk, travertin, etc., and as a secondary constituent it is found in rocks of all ages. The action of intense heat on the lime (carbonate of calcium) contained in calcareous stones expels the carbonic acid, and the lime is changed into **quicklime,** which forms the principal ingredient in the manufacture of every variety of cement used for uniting stones artificially. Calcareous stones are readily decomposed by the acids in the atmosphere, and are disintegrated by the freezing of water in the pores; they are, therefore, not durable.

SILICIOUS STONES

23. Granite, Gneiss, and Syenite.—Granite, gneiss, and syenite are commonly called *granite* by builders, owing to the great resemblance of their external appearance. They differ considerably, however, and care must be taken not to infer the nature of any particular stone from its trade or local name. Many of the so-called granites have no real claim to this designation. Granite and gneiss differ rather

in the aggregation of their constituent elements than in any other essential particular.

24. Granite is an igneous rock, ordinarily composed of quartz, feldspar, and mica, and owes its origin to the cooling and crystallization of matter under circumstances of heat and pressure that do not obtain in the case of lavas ejected on the surface in a molten state. The colors of granite are white, grayish-white, yellowish, reddish, rose, flesh color, or deep red, and occasionally green. This stone is distinguished by its even and brilliant fracture, its pearly luster, and its outline, which is seldom regular but in which may be recognized rectangles and parallelograms. Granite varies in quality according to the proportions of its components and their manner of aggregation. The hardest and most durable varieties contain a greater proportion of quartz and a less proportion of feldspar and mica. Hornblende renders granite tough and heavy. Feldspar renders it lighter in color, easier to cut, and more susceptible to decomposition by the solution of the potash contained in it. Mica renders it friable.

The granites are among the most valuable of the building stones, and are extensively used in important works. They can be readily quarried, and, by reason of the lack of grain in the stone, blocks can be obtained of any size. On account of its great hardness, however, granite is difficult to work, and therefore very costly if the stone is to be cut.

The durability of granite depends on the quantity of quartz present and on the nature of the feldspar. Potash feldspar is less durable than lime or soda feldspar. Mica, being easily decomposed, is an element of weakness. An excess of lime or soda in the mica or feldspar hastens disintegration; so does an excess of iron. Stones showing large and dark iron stains should be rejected for outside work. Fine-grained granite weathers better than that of coarser grain. When mica predominates, granite passes into gneiss.

25. Gneiss is similar to granite in composition, but differs from it in being stratified. It is less strong and

durable. It occurs in the neighborhood of granite, in strata much inclined, bent, and distorted.

26. The term **syenite** is usually restricted by modern authorities to a rock that is an aggregate of orthoclase and hornblende; in other words, a granite in which the quartz has disappeared and all or nearly all the mica has been replaced by hornblende. The syenitic granites are darker, tougher, and more compact than the ordinary granites. Syenite is durable if its feldspar constituent is not too easily decomposed by the removal of its potash when exposed to the weather. For this reason, it should be carefully tested before it is used.

27. Trap Rocks.—The term **trap,** derived from the Swedish word *trappa*, "a stair," is generally applied to a large variety of dark-colored, igneous, unstratified rocks that occur in large tabular masses rising one above another in successive steps like stairs. These rocks consist chiefly of hornblende, lime, feldspar (labradorite), and augite, with some magnetic and titanic iron. The predominance of one or the other of these minerals gives rise to many distinctive names, as *greenstone*, *olivine*, etc. The color varies, being dark gray, dark green, or nearly black, according to the varying proportions of the different constituents. The texture is usually so fine and close-grained that the character of the structure cannot be determined by the naked eye.

The trappean rocks are exceedingly dense, hard, and durable; but, owing to the difficulty of securing large blocks, because of the numerous joint planes intersecting them, the great cost of working, and their usually somber and unattractive appearance, they are not much used for structural purposes. However, as they split and break easily, trap rocks are extensively used for the aggregate in making concrete, for paving blocks, and for the construction of macadamized roads, for which purpose their fine texture peculiarly fits them.

28. Sandstones are stratified rocks consisting of grains of sand (small crystals of quartz) cemented together by

silicious, ferruginous, calcareous, or argillaceous material. From the nature of the cementing material, the rocks are variously designated as **ferruginous, calcareous,** etc. The cementing material determines the hardness, strength, and durability of sandstones; if it is one that decomposes readily, as in the argillaceous and calcareous varieties, the whole mass is soon reduced to sand. When composed of nearly pure silica and a good cementing medium, sandstones are as durable as granite and very much less affected by the action of fire. When quarried, sandstones are usually saturated with **quarry water** (in this case a weak solution of silica), and are very soft; but, on exposure to the air, they become hard by the precipitation of the soluble silica.

The color, too, of sandstone depends on the cementing material. A stone composed exclusively of grains of quartz is snow-white. The various shades of red and yellow are produced by the iron oxides; the purple tints are due to oxide of manganese; the gray, blue, and green tints are produced by iron in the form of ferrous oxide, carbonate, or silicate; the brown color is produced by the hydrated oxide of iron, and the stone possessing it is called **brownstone.**

Sandstones are, in general, porous and capable of absorbing much water, but they are injured comparatively little by moisture, except when placed with the layers set on edge, in which case the expansion of water in freezing between the layers makes them split, or "scale" off the face of the stone. When placed on the natural bed, any water that may penetrate between the edges of the layers has room to expand or escape. Sandstone containing much lime in the cementing matter decays rapidly in the atmosphere of the sea coast, and in that of towns where much coal is burned: in the former case, the lime is dissolved by muriatic (hydrochloric) acid; in the latter, by sulphuric acid. Crystals of sulphuret of iron are sometimes embedded in the stone; when exposed to air and moisture, they decompose and cause disintegration. These crystals are easily recognized by their yellow or yellowish-gray color and metallic luster.

On account of its easy working qualities, sandstone has been named **freestone** by stone cutters. A great many other names are applied to it, derived from the appearance of the stone and the uses to which it is put.

ARGILLACEOUS STONES

29. Slate is a stratified rock of great hardness and density, with a laminated structure. It splits readily along planes called *planes of slaty cleavage*. This facility of cleavage is one of the most valuable characteristics of slate, as it enables masses to be split into slabs and plates of small thickness and great area.

The color of slates varies greatly; those most frequently met with are dark blue, bluish black, purple gray, bluish gray, and green. Red and cream-colored slates are also occasionally found. Some slates are marked with bands or patches of color different from that of the general color of the stone. These marks are generally considered not to injure the durability of the slate, but they lower its quality by impairing its appearance.

Ribs, or **veins,** are dark marks running through some slates. They are always objectionable, but particularly when they run in the direction of the length of the slate, which is very liable to split along the vein. These veins and ribbons are frequently soft and of inferior quality to that of the slate proper, and slates containing them should not be allowed in good work.

Although not strictly a building stone, slate is used extensively for covering steps and the roofs of buildings, for wall linings, and for sanitary purposes.

CALCAREOUS STONES

30. The limestones are all of sedimentary origin, and have for their principal ingredient carbonate of lime, combined with various minerals. The presence of these minerals gives rise to the division of the limestones into five classes, each of which is designated by the name of the predominating mineral. When clay is present, the stone is called **argillaceous limestone;** when silica predominates, **silicious limestone;** when it contains iron, **ferruginous limestone;** when magnesia is present to the extent of 15 per cent., **magnesian limestone.** When the carbonate of lime and the carbonate of magnesia are combined in equal proportions, the stone is called **dolomite.** Limestones are either *granular* or *compact.*

31. Granular limestone consists of carbonate of lime in grains, which are in general shells or fragments of shells cemented together by some compound of lime, silica, and alumina, and often mixed with a greater or less quantity of sand. Granular limestone is always more or less porous. It is found in various colors, especially white and light yellowish brown. In many cases, it is so soft, when first quarried, that it can be cut with a knife; it hardens, however, on exposure to the air. The variety of granular limestone called *oolitic*, from the appearance of the stone, which is that of egg-shaped grains cemented together, is one of the most important of the limestone group, and is extensively quarried and widely used for building purposes. The term oolitic is derived from the Greek words *oon*, "an egg," and *lithos*, "a stone." The small round particles of which oolitic limestone is composed resemble the eggs, or roe, of a fish. Each grain is usually of concentric structure, the carbonate of lime enclosing a particle of sand or some other substance of either animal or vegetable origin.

32. Compact limestone consists of carbonate of lime, either pure or mixed with sand or clay. This limestone is generally devoid of crystalline structure, and has a dull earthy appearance and a dark-blue, gray, black, or mottled color.

In some cases, however, it is crystalline and full of organic remains: it is then known as **crystalline limestone.**

The compact limestones are easily worked with the saw and hammer, resemble light granite in appearance, and are much used for building purposes. The variety called **shelly limestone** consists of fossil shells, cemented together and sufficiently hard to take a polish; it is much used for interior ornamentation. The condition of the minerals combined with the lime also furnishes a basis for distinguishing names. The stone is called **hornstone** when very fine-grained silica is present; **cherty,** when the silica is in the form of rounded masses or nodules; **ironstone,** when the amount of iron and clay is greater than the amount of lime; **rottenstone,** when the ironstone is decomposed; **hydraulic limestone,** when the rock is composed of lime, silica, and clay in nearly equal proportions.

33. The limestones form an important and useful group of stones, but not all are suited for structural purposes; some are too friable, and others too brittle. The compact and granular varieties, however, are generally suitable for masonry. Their durability depends mainly on the texture; when this is compact, the stone is very durable, except when exposed to the acid vapors of cities. Nearly all the varieties are attacked by sulphuric acid, which forms a soluble sulphate of magnesia easily washed away.

34. Marble is the purest form of carbonate of lime (except stalactites). The name marble is generally applied to any limestone that can be polished. Marbles are found in a great variety of colors and degrees of fineness. The principal use of marble is for interior decoration and ornamental work.

PROPERTIES OF BUILDING STONES

35. Color.—As a rule, the chemical constituents of a stone determine its color. The color of granites, however, is affected by the action of light on the feldspars, which, when clear and glassy, absorb the light, making the rock apparently darker than when the feldspars are white and

opaque and reflect the light. The compounds of iron are the principal coloring substances. The brownish or reddish hues are due to the free oxides of iron, while the bluish or grayish hues are caused by the carbonates or the sulphides. The absence of iron in any of its forms is usually indicated by the white or nearly white color of the stone. The permanency of the color of the stone depends on the form in which the iron is found. The decidedly red color may generally be considered permanent. The blue and the black colors of marbles and limestones are largely caused by the presence of carbonaceous matter, usually of vegetable origin.

36. Durability.—In order that a stone may be **durable,** it must be free (1) from internal decomposing elements, as ferric oxide in the hydrated form, sulphide of iron, feldspar showing incipient decay, carbonate of calcium, protoxide of iron, excess of non-crystalline alumina, and organic matter; (2) from cavities or fissures, either empty or filled with liquids; (3) from irregular lamina and seams, or patches of soft material not thoroughly cemented together, technically called **drys;** (4) from veins, called **crowfoots,** containing uncemented material; and (5) from lack of uniformity in hardness, texture, solubility, and porosity.

37. Regarding the durability of the common varieties of building stone, the following rough estimate, based on observations made in the city of New York, indicates the number of years a sound stone may be expected to last without being discolored or disintegrated to such an extent as to require repairs:

Name of Stone	Life of Stone Years
Coarse brownstone	5 to 15
Compact brownstone	100 to 200
Limestone	20 to 40
Granite	75 to 200
Marble	40 to 200

38. Strength.—In ordinary buildings and engineering structures, stones are generally under compression. Occasionally, they are subjected to cross-stresses, as in lintels

over wide openings. They are never subjected to direct tension. As a general rule, a stone should not be subjected to a greater compressive stress than one-tenth of the ultimate crushing strength, as found by experiment.

TABLE I

APPROXIMATE AVERAGE CRUSHING STENGTH OF STONES

Stone	Crushing Strength Pounds Per Square Inch
Granite . .	15,000
Sandstone .	10,000
Limestone .	13,000
Marble . . .	14,000

The resistance to crushing varies within wide limits, owing to the great variety in the structure of the stones; the method of preparing and finishing the test pieces also affects the results; hence, the great variations found in the values given by different experiments. Table I shows the average resistance to crushing of the principal stones. These values are here given that a general idea of the strength of stone may be had; they are not the exact values likely to be used in the design of an actual structure. Every structure of importance is built according to specifications, in which the strength of the materials is stated; and the materials must be so selected that, on being tested, they will show the specified strength.

TABLE II

AVERAGE MODULUS OF RUPTURE

Stone	Modulus of Rupture Pounds Per Square Inch
Granite . . .	1,800
Sandstone . .	1,200
Limestone . .	1,500
Marble . . .	2,160

The strength of a stone to resist rupture when employed as a beam or lintel is much less than its strength to resist a crushing stress. Table II shows the average modulus of rupture of the different stones.

39. Hardness.—The **hardness** of a stone depends on the hardness of its mineral constituents and on their state of

aggregation. The component minerals may be hard, but the stone itself will be soft if the particles do not adhere strongly to one another. Thus, some of the softest sandstones are composed of quartz, which is a hard mineral, but the grains are so weakly cemented together that the stone as a whole is soft. Hardness does not imply that a stone is durable: many hard stones are more affected by atmospheric agencies than those of a softer texture whose chemical composition is of a more durable nature.

40. Density, Weight, and Absorptive Power.—The properties, density, weight, and absorptive power are closely related. The density depends on the contiguity or closeness of the aggregation of the mineral grains forming the stone. The closer the grains, the denser and heavier will be the stone, and the less will be the amount of interstitial space; consequently, among stones having the same mineral composition, but differing as to structure, the one having the closest and most compact structure will be the densest, heaviest, and least absorptive.

41. Resistance to Fire.—In the fierce conflagrations that occur in cities, the stone walls of structures are frequently subjected to intense heat. While stone is an excellent non-conductor, it is not as a rule so durable when subjected to intense heat as brick. The severest test to which a stone can be subjected in a fire is for it to be heated intensely and then cooled by the sudden application of water from a fire-hose. This rapid change of temperature causes the heated exterior layer of the stone to contract more rapidly than the cooler interior, and from many stones under this condition large pieces will crack and break off; this finally causes the entire destruction of the stone. The silicious sandstones are the least destructible by fire, while the granites and conglomerates are probably the most affected by intense heat and the sudden cooling incident to the application of water. Limestones are very refractory in temperatures less than 1,000°, and at this temperature are not liable to

deterioration by sudden cooling; but above this temperature, they may be reduced to quicklime, which crumbles and falls away after a few weeks' exposure to the air.

DISINTEGRATION OF STONE

DISINTEGRATING AGENTS

42. The disintegration or decay of stone is commonly referred to as **weathering,** and is caused by agents of three kinds; namely, *physical* or *mechanical*, *chemical*, and *organic*. The **mechanical agents** are heat and cold, air in the form of wind, and water in the form of rain and ice. The **chemical agents** are the various acids present in the atmosphere. The **organic agents** are vegetable growths that thrive in damp and shady places, and marine insects or boring mollusks, which perforate the stone between high- and low-water mark.

PHYSICAL AGENTS

43. Heat.—High temperature causes expansion in a stone, and cold or a decrease in temperature causes contraction; hence, there is a continual slight movement among the particles of the stone, which may destroy their cohesion, and thus produce a slow and gradual disintegration.

44. Air and Water.—Air acts mechanically in the form of wind, especially when it carries dust; it erodes the surface and removes small particles, thus exposing new surfaces to be acted on, much in the same way as a sand-blast apparatus. Air and rain together act very energetically. Rain alone has a slight mechanical effect when simply falling on the stone and washing loose particles away.

Water penetrates into all rocks, no matter how dense or compact they may be, and, when it freezes, it expands and tends to split the stone. In the formation of ice, water expands in the proportion of 100 to 109; that is, a volume of water occupying 100 cubic inches before freezing occupies

109 cubic inches after freezing. The pressure exerted by this expansion is equal to 150 tons per square foot, which is sufficient to split the strongest rocks.

CHEMICAL AGENTS

45. As a rule, air and water act together to cause: (1) **oxidation,** or rusting of the iron particles present in the stone; (2) **deoxidation,** or the changing of ferric oxide into a ferrous oxide, which is caused by the presence of an organic acid or by continual moisture; (3) **hydration,** or the process whereby an oxide absorbs water—this change occurs only in the presence of continual moisture, as in bridge piers and abutments; (4) **solution,** or the dissolving of the constituents that are soluble in water. Pure water alone has but little effect on the constituents of stone, but rain water containing acids is a powerful solvent of mineral matter. The stones that are peculiarly susceptible to this solvent action are the limestones, the calcareous sandstones, and the granites containing feldspar.

46. Carbonic acid, which is contained in the atmosphere to the amount of about 400 parts of acid to 1,000,000 parts of air, has, when combined with water, a corroding action on the calcareous and magnesian carbonates, whether they form the principal constituents of the stone or are only present as cementing materials. It transforms the insoluble earthy carbonates of lime and magnesia into bicarbonates, which are soluble in water and can therefore be washed away. On granite, it acts by eliminating the alkaline constituents in the form of carbonates, leaving a friable residue of hydrated silicate of alumina, which contains the unaltered particles of quartz and mica. In the case of the greenstones and basalts, it acts on the iron, changing it to a ferric oxide or ferric hydrate, and dissolves out the lime, leaving a loose, friable, and bulky stone of a red or brown color. Sandstones containing iron are disintegrated by the solution and washing away of the iron.

47. **Nitric acid** is frequently present as a constituent of the atmosphere; its destructive action is exerted on the calcareous and magnesian stones.

48. The **sulphurous acids,** which result from the combustion of coal, and are present in the atmosphere of cities to an extent as great as 25 parts in 100,000, have a marked destructive influence on all stones. The destroying effect consists in the formation of soluble sulphites and sulphates with the different bases contained in the stones. On granite, the action is especially severe: the feldspar is attacked, the potash, soda, or lime is dissolved out, and in time the stone becomes filled with small holes.

Sulphuric acid not only corrodes and renders soluble the earthy carbonates (in which respect it resembles carbonic acid), but, forming with magnesia a readily crystallizable salt (the sulphate of magnesia), which is remarkable for the large proportion of water of crystallization that it fixes, it causes a mechanical destruction of the stone, similar to that produced by the solidification of water.

CONDITIONS AFFECTING DISINTEGRATION

49. The disintegration and decay of stone by the inanimate agents described above are frequently materially aided by many forms of life, such as bacteria, mosses, worms, etc., which are all in a sense destructive agents; their presence gives rise to small amounts of organic acids, which exercise a corrosive influence.

50. The disintegration of stone is hastened or retarded by the methods employed in quarrying, seasoning, dressing, and setting. In quarrying, the excessive use of high explosives, by reason of the heavy concussion produced, shakes and shatters the cohesion of the particles composing the stone, and causes incipient cracks and flaws that make the stone more permeable to moisture, and thus facilitate the destruction by freezing and chemical action; hence, stones quarried by channeling and gadding are preferable to those blasted or wedged out.

Before a stone is placed in a structure, the interstitial moisture, called **quarry water,** or **sap,** must be removed by evaporation; this process is termed **seasoning,** and should be effected by exposing the stone to the drying action of the atmosphere for some months under cover. If the stone is not seasoned, the quarry water will be alternately frozen and thawed during a series of years, finally causing the destruction of the stone.

51. The life of a stone is also dependent on the style of finish of the exposed face of the stone. A smooth or polished surface aids in prolonging the life by facilitating the rapid discharge of rain water. The methods employed, too, for dressing the stone affect its life. Minute fissures that render the stone more susceptible to atmospheric influences are produced by the impact of the blows of the hammers and other tools employed; hence, stones sawed to the required dimensions are more durable than those hammered and broken to size.

52. The position of the stone in the structure affects its ability to resist disintegration. When stratified stones are placed on edge, water filters in from above, and on freezing causes the stone to scale off; hence, laminated stones should be set with their lamina horizontal. When the mortar joints are not properly pointed, water filters in and decay quickly follows. The portions of a structure most liable to early decay are those that are partly protected by projections, as under cornices, belt courses, window sills, etc., on which the rain water slowly falls or drips. As a protection from this source of decay, the under surface of all projections should have a narrow groove extending their whole length; this groove is called a **drip.** The water that collects on the upper surface of the projection flows over the upper edge and over the face to the under side, where its further progress is interrupted by the drip, and from there falls to the ground.

PRESERVATION OF STONE

53. For preventing the destruction of stone, many preparations are used, among them being paint, coal tar, oil, beeswax, resin, paraffin, soap, soluble silicates of potash. soda, salt, and fluosilicates of magnesia, zinc, and alumina. All these methods are expensive, and there is no evidence to show that they afford permanent protection to the stone.

SELECTION AND EXAMINATION OF STONE

54. General Considerations.—In order to form a judgment and determine the structural value of any stone, the stone must be examined to ascertain if it possesses the requisites described in Art. **36** as being essential to a good stone. The enduring qualities of stone from quarries that have been long worked may be ascertained by an inspection of the structures that have been erected with it. If it shows no signs of disintegration, it is considered that the stone is a durable one; however, this is not a guarantee that the same quarry will in the future yield a durable stone, because it frequently occurs in the case of limestone and sandstone that adjacent beds vary widely in color, texture, and enduring qualities.

In the examination of stones, the following general facts should be borne in mind:

1. That a fine, uniform-grain, compact texture, and deep color indicate strength and hardness.

2. That when the grain, color, and texture are the same, the heaviest stones are the strongest.

3. That the strength does not always increase with the specific gravity.

4. That great hardness is objectionable when the stone is to be worked with a chisel, owing to the labor required, and is objectionable in stones that are to be used for stairs, floors, or pavements, as they wear smooth and become polished by attrition.

5. That brittleness is a defect that frequently accompanies hardness and prevents the stone from being worked to a true surface.

6. That a low rate of absorption indicates enduring qualities.

7. That stones quarried in winter or wet seasons are likely to have slight tenacity when dried and to remain always susceptible to the effects of moisture.

8. That stones to be carved should be quarried in the spring, because they retain the quarry water longer and are consequently less injured by working.

9. That stones quarried in the summer should not be allowed to lie in the rain for a great length of time, because they will dry too quickly and be likely to become shaky and friable.

10. That stones quarried from above the water level are more durable than those quarried from below.

11. That the buff, yellow, and reddish-brown colors are the most stable.

12. That non-porosity does not always indicate durability, for many stones that absorb water permit also of its rapid evaporation; such stones are likely to prove more durable than those that absorb less water but retain it longer.

13. That stones showing a streaked appearance and lack of uniformity in color are usually composed of minerals of various degrees of hardness and some of which may be soluble; such stones are not likely to weather well. The soluble minerals will be dissolved and washed away, leaving the stone a mass of small pits or hollows. If the soluble mineral is in the form of streaks or veins, the stone will be grooved, fissured, or channeled. The presence of small fossils or shells has a detrimental influence; being calcareous in nature, they are decomposed by the acids of the atmosphere.

14. That rocks containing iron pyrites in the form known as *marcasite* decay rapidly through oxidation.

15. That a new fracture should have a clean, bright, sharp appearance. A dull earthy appearance indicates a stone likely to early decay.

When an important structure is to be erected, the quarry and building inspectors should not be wholly relied on in selecting the stone; their opinion should be supplemented

by the laboratory investigation which comprises the *optical*, the *chemical*, and the *mechanical* examination.

55. Optical Examination.—The structure of a rock is examined optically in two ways: (1) by the unaided eye, the structure that can be so distinguished being technically termed **macroscopic structure**; (2) by the microscope, the structure that can be so distinguished being termed **microscopic structure.** For the microscopic examination, a thin chip of the rock is ground with emery until it becomes transparent; it is then mounted on a microscopic slide and examined. The microscopic examination reveals more accurately the physical composition and character of the structure than any other test. The microscope shows the size and shape of the component particles, their relative closeness, and the character and composition of the cementing material. By its use are discovered defects that otherwise would escape unnoticed, such as cracks, cavities, fractures, and incipient disintegration.

56. Chemical Examination.—The chemical analysis determines both qualitatively and quantitatively the chemical constituents of the stone. Examined qualitatively, the character or kind of the substances composing the stone is determined; while the quantitative analysis shows the proportions of these substances.

57. Mechanical Examination.—The mechanical examination of a stone furnishes data from which a fair estimate of the durability may be made. It includes the determination of the resistance to crushing and transverse stresses, and the resistance to abrasion, heat, and cold. In making these tests, the object is to impose on the stone as nearly as possible conditions that in a few hours' or weeks' time will approximate the effect produced by actual use during a lapse of years. This examination is called **testing,** and is made as described in the following articles.

METHODS OF TESTING STONE

58. Absorptive Power.—Few of the properties of a stone are of greater importance than the **absorptive power,** since it is largely through the freezing of the absorbed water that the majority of stones are destroyed. The absorptive power of a stone is usually ascertained by two tests—one to determine the absorption from a moist atmosphere, and one to determine the amount of water absorbed through actual soaking. The first test is performed by keeping samples of the stone in the cells of a hot-water bath for several days to expel the microscopic moisture, after which they are cooled in desiccators over sulphuric acid, and weighed. They are then placed on shelves in a cylinder, the mouth of which is sealed with water, after the manner of a gas holder. The cylinder and the samples are kept for several weeks in a temperature ranging between 60° and 70° F., the water being replenished from time to time so as to maintain a constant closure of the mouth of the cylinder. At the end of the test period, the samples are weighed. The increase in weight shows the amount of absorption.

TABLE III

ABSORPTIVE POWER OF STONES

Stone	Absorptive Capacity Per Cent.
Granites . . .	.066 to .155
Sandstones . .	.410 to 5.480
Limestones .	.200 to 5.000
Marbles . . .	.080 to .160
Trap	.000 to .019

To ascertain the amount of water absorbed by soaking, the specimens of stone are dried and weighed, then immersed in water for 24 hours, removed, and weighed again; the increase in weight will be the amount of absorption. This is usually expressed in a percentage of the weight of the dry stone. An absorption of more than 3 per cent. is regarded as detrimental.

The average percentage of water absorbed by stones is shown in Table III.

59. The amount of water absorbed depends largely on the density of the stone; a dense stone absorbs less than a porous stone. Stones that have already begun to decompose absorb a much larger quantity of water than those fresh from the quarry. A low absorption is generally considered as indicating a good quality; still it does not follow that a stone that absorbs a small amount of water will suffer the least through the action of frost, for the reason that a porous stone of coarse structure will dry more rapidly than one of a firmer grain and open texture, and will permit the expansive action of freezing water to find relief without forcing apart the particles of which the stone is composed. Hence, a high rate of absorption is more detrimental to a fine- than to a coarse-grained stone.

60. Resistance to Freezing.—To ascertain the probable ability of a stone to resist the expansive action of freezing water, several tests are recommended, such as exposing the stone to the action of freezing mixtures. **Brard's test,** which consists in boiling weighed samples in a concentrated solution of sulphate of soda, is considered the best. The soda in crystallizing expands, as does water when freezing. After each boiling, the stone is removed from the solution and hung up to dry. The operation is repeated daily during a period of 4 weeks, after which the stone is dried and weighed, and the difference in weight and the general appearance are noted.

61. Resistance to Abrasion.—The resistance to abrasion is ascertained by placing cleaned and weighed fragments of the stone in a metal cylinder and revolving it at the rate of about 30 turns a minute until 10,000 revolutions have been made; as the cylinder revolves, the stones are rolled against one another, and the edges are gradually broken off, the particles thus separated forming a dust. When the required number of revolutions has been reached, the stone is removed from the cylinder and weighed. The difference between the two weighings represents the loss by abrasion. The ability of stones to resist abrasion is

compared by the ratio of the weight of the dust worn off to the original weight of the stone, and the loss is expressed as **per cent. of wear.** Thus, if the original weight of the stone placed in the cylinder was 20 pounds, and the weight of the dust produced was 5 pounds, the loss would be expressed as 25 per cent. of loss or wear.

Regarding the ability of stones to resist abrasion, it may be stated generally that the igneous rocks lose from 2 to 10 per cent.; the range of loss in limestone rocks varies from 10 to 35 per cent.; flints lose from 8 to 26 per cent.; and sandstones lose about 14 per cent.

A stone that, when tested for abrasion, yields from 2 to 6 per cent. of dust is considered to be an excellent stone for road construction; 10 per cent. of dust indicates a weak stone, which will produce a large amount of mud.

The resistance to abrasion by wind-blown sand is ascertained by subjecting weighed samples of the stone to the action of a sand blast operated under a given pressure for a specified time, at the end of which the sample is weighed to ascertain the loss.

62. Crushing Strength.—The **crushing strength** of a stone is ascertained by subjecting cubical specimens accurately dressed to form and dimensions to a measured force applied in a suitably constructed machine, until they are crushed.

63. Transverse Strength.—To ascertain the **transverse strength,** prisms 1 inch square and from 6 to 12 inches long are supported at each end and loaded in the center until fracture takes place. The breaking load thus found is the modulus of rupture of the stone, and is employed to ascertain the breaking load of any stone under transverse stresses (see *Strength of Materials*). Owing to the uncertainty regarding the strength of stone, a working strength of from 10 to 20 per cent. of the ultimate strength is used.

64. Resistance to Impact.—The **resistance to impact,** or to the action of blows, is very important in connection with stones used for the wearing surface of roads.

It is determined by the number of blows necessary to fracture a piece of the stone of a given size. The test employed is called the **impact test**, and is effected by taking pieces of the stone broken to a size that will pass in all directions through a ring having a diameter of $2\frac{1}{2}$ inches, and subjecting them to repeated blows of a falling weight operated by a machine resembling a pile driver. The hammers range in weight from 15 to 100 pounds, and the height of fall ranges from 10 to 36 inches. The sample of stone to be tested is placed on an anvil, and the hammer is raised and allowed to fall repeatedly, until the stone is reduced to fragments not larger than $\frac{1}{4}$ inch. The number of blows necessary to accomplish this result is automatically registered. A stone fit for roadmaking should withstand at least 200 blows of a 15-pound hammer falling 10 inches without being reduced to fragments less than $\frac{1}{4}$ inch. The number of blows of a 15-pound hammer, falling 10 inches, required to reduce a sample of quartz to dust is about 200; and for trap, the number is from 900 to 1,000.

65. Cementing Capacity.—The **cementing capacity** of a stone, or its ability to form a mortar-like paste, is another important property of stones used for roadmaking. It is ascertained by taking the dust produced in the abrasion test, and forming a paste by mixing it with water; this paste is placed in a metal die and compressed under a pressure of 1,000 pounds per square inch into a briquet having a diameter and length of about 1 inch. The briquet is laid aside for 2 weeks to dry, and is then broken by being struck a number of light blows with a hammer weighing about 2 pounds and falling about $\frac{1}{2}$ inch; the number of blows required to break the briquet is taken as the measure of the cementing value. Experiments conducted to ascertain the cementing qualities of different stones show that quartzites, granites, gneisses, and marble possess very little cementing power, only two or three blows being necessary to break the briquet; and that limestones and trap rocks have considerable cementing power, and require thirty to

forty blows to break the briquet. The cementing quality appears to be due to the presence of oxide of lime or iron, and also to small particles of clay formed by chemical disintegration of feldspar.

66. Permanence of color is ascertained by submitting samples placed in an air-tight vessel to the action of the fumes of nitric, hydrochloric, and other acids, for a period of 7 or more weeks, at the end of which time the stones are washed and any change in color is noted.

67. Resistance to Acids.—The effect of the acids contained in the atmosphere is determined by immersing samples of the stone for several days in water that contains 1 per cent. of the acid whose action it is desired to ascertain, and agitating frequently.

68. Specific Gravity.—The determination of the specific gravity of stone affords a convenient method of ascertaining the weight per cubic foot. This determination is made by carefully weighing a small piece of the stone in the air and then weighing it in water. The result obtained by

TABLE IV

SPECIFIC GRAVITY AND WEIGHT OF STONE

Kind of Stone	Specific Gravity		Weight Pounds per Cubic Foot	
	Minimum	Maximum	Minimum	Maximum
Granite . . .	2.60	2.80	163	170
Sandstone . .	2.23	2.75	137	170
Limestone . .	1.90	2.75	118	175
Marble . . .	2.62	2.95	165	179

dividing the weight in air by the difference between the weight in air and the weight in water is the specific gravity, which multiplied by 62.5, the weight in pounds of 1 cubic foot of water, gives the weight of 1 cubic foot of the stone. (See *Hydrostatics.*)

When it is desired to ascertain the specific gravity of porous stones, or those that absorb much water, the specimen is first weighed dry, then immersed in water, and, when thoroughly saturated, removed and weighed, again immersed, and weighed while under water. The quotient obtained by dividing the dry weight by the difference between the weights of the saturated stone in air and under water will be the specific gravity.

Table IV contains the specific gravity and weight per cubic foot of the usual building stones.

69. Resistance to Fire.—The power of a stone to resist the action of high temperatures is ascertained by heating samples to a red heat in a muffle furnace and observing the effect. When slightly cooled, the heated samples are plunged into cold water, and the effect in producing cracks or crumbling is noted.

QUARRYING

70. General Methods.—In quarrying stone, the first operation is the stripping or removing of the soil or earth covering; the second is the forming of a working face, by rinsing the top stone, which is usually earth-stained and weathered. When a sufficient space of sound rock is exposed, the quarry is ready to produce stone for structural purposes.

The methods employed for quarrying or loosening the rough blocks of stone vary with the character of the rock. The object aimed at is to produce large and well-shaped blocks with the least outlay of time and money, and to avoid as far as possible the use of explosives. In quarrying stones that occur in thin layers, the loosening is effected by the use of hand tools, such as hand drills, plugs and feathers, wedges, etc. When the layers are of considerable depth, holes are drilled at regular intervals along the lines bounding the block to be loosened, the number of holes depending on the character of the rock and the size of the block. When the holes have reached the required depth, they are charged with some explosive, as black powder or dynamite. The former of

these explosives is to be preferred, for the reason that it causes less injury to the stone. Dynamite has a tendency to shatter and break the stone in many directions, and it affects the texture in the same manner as the blows of a hammer. For drilling the holes, hand or steam drills are employed.

The method just described is the method practiced in quarrying granite and the harder rocks.

71. In loosening marble, limestone, and sandstone, machines called **channeling machines** are used. They cut a continuous groove or channel along the sides of the block to be detached. When the rock is in well-defined layers, it is only necessary after the channels are cut to separate the block from its bed by wedges. When the rock is not in layers, the block to be removed must be **undercut** after the channels have been cut around it. This is done by a machine called a **gadder,** which cuts a series of horizontal holes at the required depth, after which the stone is split out by the use of wedges or light charges of powder. The method of channeling, gadding, and wedging is extensively employed, and is very economical and expeditious.

72. Plug and Feathers.—In many quarries, very large masses of stone are broken out from the bed by drilling and blasting, and are then broken to the smaller sizes required for use by a method called **plug-and-feathering.** This method is also used for breaking up boulders. The feathers, shown in Fig. 1 (*a*), are two half-round pieces of iron tapering almost to a point at one end. They range in length from 3 to 12 inches, and are used to provide a smooth path for the plug, and prevent it from sticking in the stone. The plug, shown in Fig. 1 (*b*), is a truncated wedge of steel; its length is a few inches greater than the depth of the holes in which it is to be used.

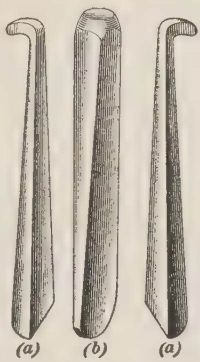


FIG. 1

The method of using the plug and feathers is to drill a row of holes, ranging from $\frac{3}{8}$ to 1 inch in diameter, at distances varying from 6 to 8 inches apart, and to a depth varying from 4 to 6 inches, along the line in which it is desired to have the stone split. A plug is placed between two feathers and inserted in each hole; the plugs are then gradually driven in simultaneously by light blows of the hand hammer until the stone splits.

STONE CUTTING AND DRESSING

73. Tools Used in Stone Cutting.—The principal hand tools used by the stone cutter in cutting or dressing stone are as follows:

Fig. 2

The **ax, or peen hammer,** Fig. 2, is used for hewing the surfaces of the stone to a uniform plane and for cutting drafts around the edges; it is about 10 inches long, and has two cutting edges. Fig. 23 shows the appearance of an axed stone.

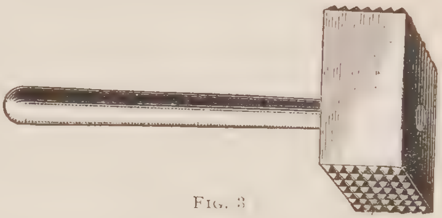


Fig. 3

The **bush hammer,** Fig. 3, is a square prism of steel, from 4 to 8 inches in length and 2 to 4 inches square. The ends are cut into a number of pyramidal points that vary in number and size according to the fineness of the work required. The appearance of bush-hammer dressing is shown in Fig. 24.

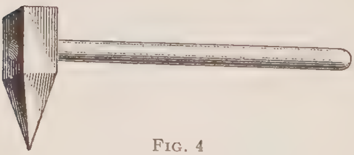


Fig. 4

The **cavil,** Fig. 4, has one blunt and one pyramidal or pointed end. It is about 12 inches long and 3 inches wide, and weighs from 15 to 20 pounds. It is used to roughly shape the stone for transportation.

The **crandall**, Fig. 5, consists of a wrought-iron or steel bar, having one end flattened for a length of about 4 inches and a width of about 1½ inches; in this flattened end there is cut a slot, ⅜ inch wide and 3 inches long, in which are inserted ten bars pointed at each end; these bars are made of ¼-inch square steel and are fastened by a key. This tool is used on sandstone after the tooth ax. Fig. 22 shows the appearance of a crandalled stone.

The **chisel**, Fig. 6, is made of round steel varying in diameter from ¼ to ¾ inch, and has a length of about 10 inches. One end is formed into a cutting edge, having a width varying from ¼ inch to 2 inches, according to the work on which it is used. This tool is for cutting drafts or markings on the face of stones. Fig. 18 shows a rock-faced stone with a chisel draft.

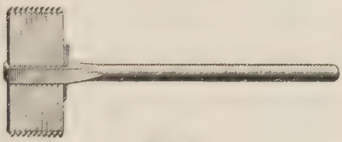


Fig. 5

Fig. 6

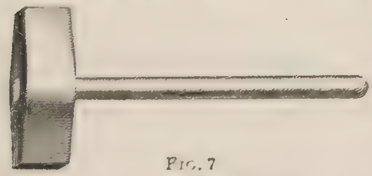


Fig. 7

The chisel is also used in producing the finish called **drove work** shown in Fig. 27. The work is done with a wide chisel, and the character or fineness is regulated by the number of blows, usually four, given the chisel to each inch in the length of the stone.

The **double-face hammer**, Fig. 7, is used in the quarry for roughly shaping the stone and knocking off irregular angles. It has two square flat faces, and weighs from 20 to 30 pounds. This tool is also called a **spalling hammer.**

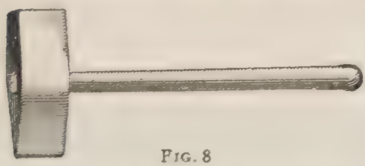


Fig. 8

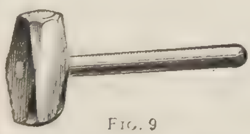


Fig. 9

The **face hammer**, Fig. 8, is used for roughly bringing stone to the required shape before applying finer tools. It

weighs about 10 pounds, and has one cutting edge and one flat face.

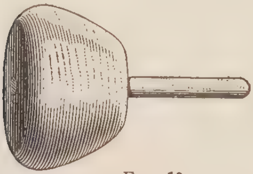


FIG. 10

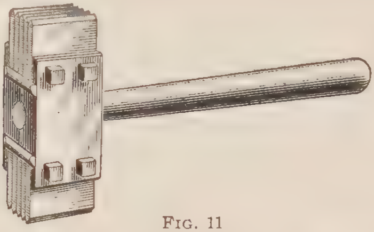


FIG. 11

The **hand hammer,** Fig. 9, is used for driving the drills and blunt-pointed tools. It is made of steel, has two opposite flat striking faces, and weighs from 2 to 5 pounds.

FIG. 12

The **mallet,** Fig. 10, usually made of hickory, has a diameter of from 6 to 12 inches. It is used to drive the point or chisel in working soft stones.

The **patent hammer,** Fig. 11, is used for dressing granite and hard limestone. It is a double-headed tool, so formed as to hold at each end a set of wide, thin chisels or blades of sharpened steel. The tool, without the teeth, is $5\frac{1}{2}$ in. $\times$ $2\frac{3}{4}$ in. $\times$ $1\frac{1}{2}$ in.; the teeth or blades are $2\frac{3}{4}$ inches wide; their thicknesses vary from $\frac{1}{12}$ to $\frac{1}{6}$ inch. The fineness of the work is designated as **4-cut, 6-cut, 8-cut,** etc., according to the number of blades used to the inch.

The **patent chisel,** Fig. 12, is used on surfaces where the patent hammer cannot be conveniently handled. In finishing surfaces with either of these tools, they should be held so that the blades are always in the same direction on the stone.

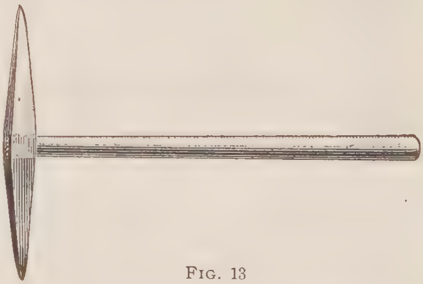


FIG. 13

The **pick,** Fig. 13, resembles the pick used in digging; the length varies from 15 to 24 inches, the thickness at the eye being about 2 inches. The pick is used for roughly dressing limestone and sandstone.

The **pitching chisel**, Fig. 14, is used with the hand hammer to bring the edges of a stone to a straight line; the stone is then said to have pitched edges or be pitched to line. This tool is made of octagonal steel, and has a beveled, instead of a cutting, edge, the dimensions of which are usually $\frac{1}{8}$ in. $\times$ $2\frac{1}{2}$ in.

FIG. 14

FIG. 15

The **point**, Fig. 15, is used for dressing or pointing off the surface of a stone, either for a permanent finish or to prepare the stone for the use of the ax. When used for a permanent finish, two classes of work are produced: **rough-pointed**, shown in Fig. 20, and **fine-pointed**, shown in Fig. 21. The point is made of round or octagonal steel, the end being sharpened to a pyramidal point, and is driven with either the hand hammer or the mallet, according to the hardness of the stone.

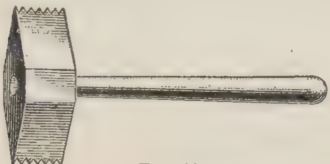


FIG. 16

The **tooth ax**, Fig. 16, is used on soft stone to bring the rough surfaces to the desired plane. It is similar to the ax, except that its cutting edges are divided into teeth, the number of which varies with the fineness of finish desired.

The **tooth chisel**, Fig. 17, is a chisel having the cutting edge divided into teeth. It is used in working soft stones, because it cuts faster than the ordinary chisel.

FIG. 17

74. Stone-Working Machinery.—In the large stone-cutting plants, mechanical appliances are employed as a substitute for the manual labor of the stone cutter. The machines comprise saws for cutting the stone into blocks; planes for cutting a smooth surface; dressing machines, which are used to prepare the surface of the stone for polishing; jointing machines for dressing the beds and joints to plane surfaces;

polishing machines for giving a final dressing and gloss to the surface of the stone; and various other machines serving different purposes. A detailed description of these machines would be beyond the scope of this Course.

75. Dressing the Stones.—Stone cutting may be practiced in various ways, the aim of all being the formation of the surfaces desired with the least possible loss of material. On receiving a rough block of stone from the quarry, the stone cutter examines it, in order to determine the purpose to which it is best adapted. He then prepares to dress the bottom **bed**, or surface. The stone is placed with the bottom bed up, all the rough projections are removed with the hammer and pitching tool, and approximately straight lines are pitched off around its edges; then a chisel draft is cut on all the edges. These drafts are brought to the same plane as nearly as practicable by the use of two straightedges having parallel sides and equal widths, and the enclosed rough portion is then dressed down with the pitching tool or point to the plane of the drafts. The entire bed is then pointed down to a surface true to the straightedge when applied in any direction—crosswise, lengthwise, and diagonally. Lines at right angles are then marked on this dressed surface enclosing a rectangle of such size as the stone will admit of being worked to, or of such dimensions as may be directed by the plan. The faces and sides are pitched off to these lines; a chisel draft is then cut along all four edges of the face, and the face is either dressed as required or left rock-faced. The sides are then pointed down to true surfaces at right angles to the bed. The stone is turned over, bottom bed down, and the top bed dressed in the same manner as the bottom. It is important that the top bed be exactly parallel to the bottom bed, in order that the stone may be of uniform thickness.

Stones having the beds inclined to each other, as skewbacks, and stones having the sides inclined to the beds, are dressed by using a beveled straightedge set to the required inclination. Arch stones have two plane surfaces or beds

inclined to each other. The upper surface, or **extrados,** is usually left rough; the lower surface, or **intrados,** is cut to the curve of the arch. This surface and the beds are cut true by the use of a wooden or a metal templet, which is made according to the drawings furnished by the engineer or architect.

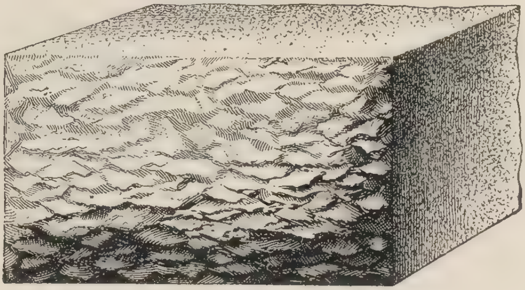


FIG. 18

76. Finishing the Faces of Stone.—For the purpose of presenting a pleasing appearance, the face of the stone is dressed or finished in many ways, according to the class of work or style of architecture in which it is to be used. The following are the finishes most generally employed:

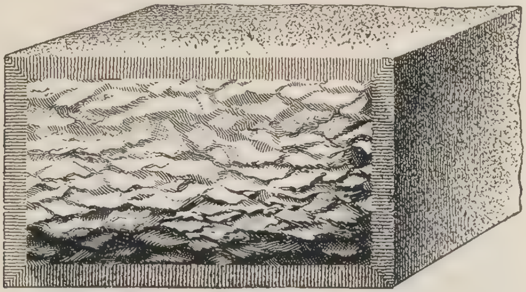


FIG. 19

Quarry-faced, or **rock-faced,** in which the face of the stone is left as it comes from the quarry. Fig. 18 shows a quarry-faced stone with **pitched edges;** that is, the stone has the arrises or edges clearly defined by a chisel line, beyond which the rock is left as it came from the quarry, or it is roughly cut away with the pitching chisel to form a projection of a prescribed amount.

Fig. 19 shows a quarry-faced stone with a dressed margin; this type is called a **drafted stone.** In it, the face is surrounded with a chisel draft, the space inside the draft being left rough. The width of the draft varies, according to fancy, from $\frac{1}{2}$ inch to $2\frac{1}{2}$ inches.

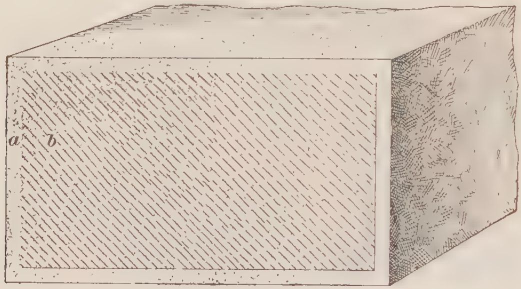


Fig. 20

In ordering stone of the above classes, the specifications should clearly state the width of draft, the width of the bed and end joints, and how far the surface of the face may project beyond the plane of the edge. In practice, the projection varies between 1 inch and 6 inches.

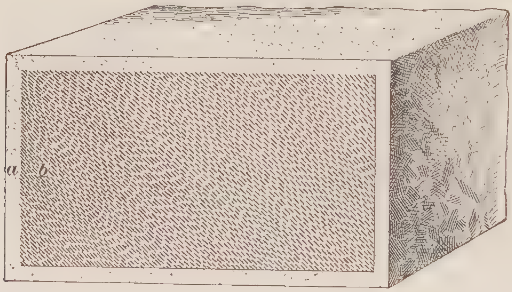


Fig. 21

77. A rough-pointed finish is shown in Fig. 20, and a fine-pointed finish in Fig. 21. In the first, the point is used so that it will leave its markings at intervals of about 1 inch; while in the second a finer tool is used and the markings are spread from $\frac{1}{4}$ to $\frac{1}{2}$ inch apart.

78. A **crandalled** finish is shown at *a*, Fig 22. The effect produced by the crandall is the same as fine-pointing, except that the dots are more regularly spread; the variations of level are about $\frac{1}{8}$ inch. When other rows at

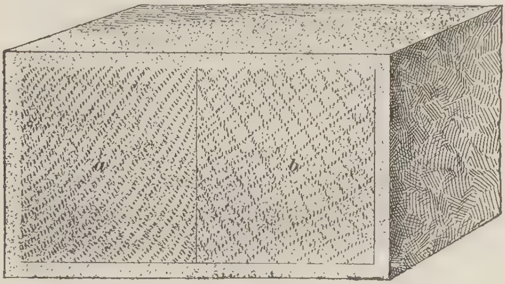


Fig. 22

right angles to the first are introduced, as shown at *b*, Fig. 22, the finish is called **cross-crandalled.**

79. **Axed** and **patent-hammered** finishes are shown in Fig. 23. They differ only in the degree of smoothness; the effect is a series of parallel lines, the distance apart of which is regulated by the number of blades used in the hammer. Precise specifications designate the number of cuts required to the inch, as 6-cut, 8-cut, 10-cut, 12-cut, etc.

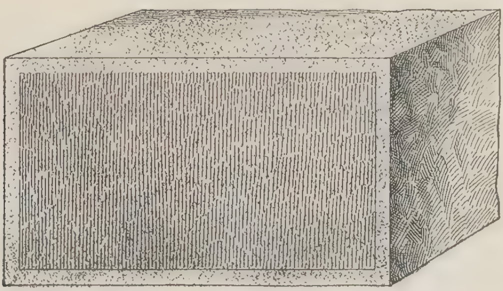


Fig. 23

80. **Bush-hammered,** Fig. 24, shows a mass of small dots corresponding to the points of the hammer. **Rubbed** and **polished** are smooth finishes produced by sawing, grinding, etc. Such finishes are used for string-courses,

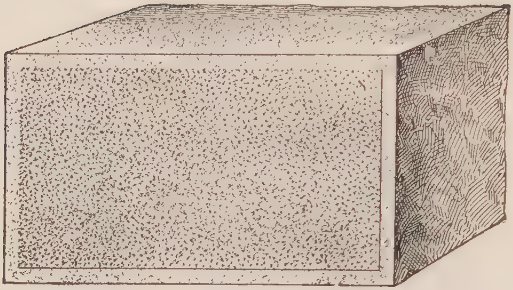


Fig. 24

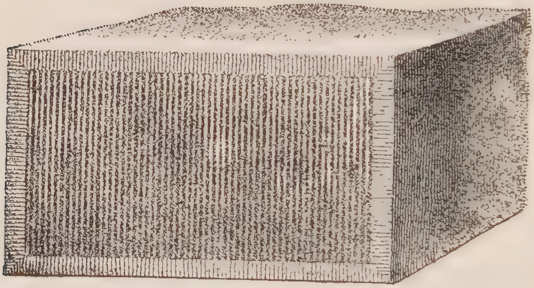


Fig. 25

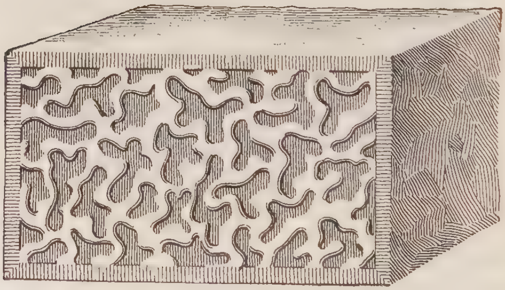


Fig. 26

door jambs, lintels, etc.; and in positions where the surface of the stone is liable to be rubbed by moving bodies, as in lock chambers.

81. Tooled work, Fig. 25, is executed with the flat chisel, and is used for sandstones and limestones. **Vermiculated work,** Fig. 26, is made with the chisel to give

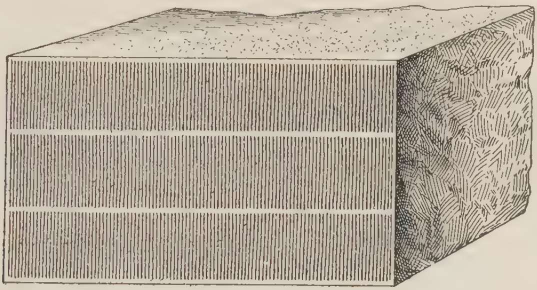


Fig. 27

the stone the appearance of having been eaten by worms. **Drove work,** Fig. 27, is similar to tooled work, except that the lines are broken.

82. Diamond-panel work is shown in Fig. 28. Two forms of this finish are used: the **raised** and the **sunken.**

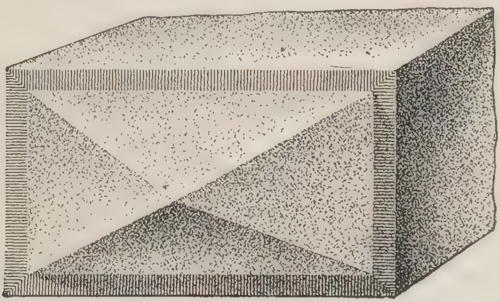


Fig. 28

In the first, the surface of the stone is cut so as to slope up from the draft line to form a raised apex in the center; in the sunken, the stone is cut away so as to slope inwards and form a sunken apex in the center. Both kinds are used for bridge quoins and similar work.

83. Inspection of Cut Stone.—The stone-cutter's shed should be frequently visited, and the stones on hand examined (1) to discover any defects that may have been overlooked in the examination of the rough stone; (2) for correctness of the dimensions; (3) for the character and quality of the workmanship. The dressing of the bed joints should receive special attention. The surface of the bed should be true to the straightedge placed in every direction across it.

Stones accidentally broken after being cut should not be allowed to be patched and used. The practice of patching is frequently followed in granite and other brittle stones. The broken pieces are glued in with melted shellac. In dry weather and while still fresh from the tool, such patches are hardly noticeable, unless near the eye; hence, they should be closely looked for. When the stone is wet by rain, the patch becomes conspicuous, and as the shellac is slowly destroyed the piece may eventually drop out.

BRICK

84. The word **brick** is generally used to describe small blocks made from clay and hardened by burning in a kiln. Until very recently, all bricks were made from clay; at the present time, however, other materials, as mixtures of lime and sand, and of cement and sand, are being employed.

CLAY BRICK

85. Clay bricks are made by submitting clay that has been suitably prepared and molded into shape to a temperature of sufficient intensity to reduce it to a semivitrified state. There are several varieties of clay brick, each one of which is used for a different purpose: as, **common brick**, employed for interior walls, the foundation and backing of exterior walls, etc.; **pressed** or **face brick**, used for the exterior face of walls; **glazed** and **enameled brick**, used for facing walls in lavatories and public places where sanitary

considerations require the use of much water for cleansing purposes; **firebrick**, used for lining furnaces and in other situations where high temperatures are to be resisted.

86. The clay from which common and pressed bricks are made consists chiefly of silicate of alumina, either alone or combined with other substances—such as iron, lime, soda, potash, magnesia, etc.—on the relative proportions of each of which the character and quality of the brick, to a great extent, depend. Iron gives strength and hardness. Silicate of lime renders the clay fusible, and causes the brick to become distorted in burning. Carbonate of lime renders the clay fusible, and, when exposed to the action of the weather, absorbs moisture and prevents the adherence of the mortar. Uncombined silica is beneficial when present in moderate quantity (one part of silica to four parts of clay); it preserves the shape of the brick and prevents shrinking, while an excess destroys cohesion and renders the brick weak and brittle.

87. The color of clay brick depends on the composition of the clay, the molding sand, temperature of burning, and volume of air admitted to the kiln. Pure clay burns white. Iron, according to its proportion, produces colors ranging from red and orange to light yellow; 8 to 10 per cent. of iron produces a dark blue or purple, and with the addition of a small amount of manganese produces black. Lime and iron produce a cream color; magnesia with iron gives yellow. Alkalies produce a bluish-green color.

MANUFACTURING PROCESSES

88. Clay brick is made both by hand and by machinery. The manufacturing operations comprise the following: (1) **Screening** the clay to remove stones and mechanical impurities; for this purpose, the clay is passed through a series of perforated metal screens vibrated by machinery. (2) **Tempering**, that is, mixing the clay with water to make a plastic mass; this is effected either in wet pans or in pug

mills. (3) **Molding,** or shaping the brick. (4) **Drying** the molded brick, which is done either in the open air, or in steam-heated drying houses, or on hot floors. (5) **Burning.** The processes of molding and burning require especial consideration.

89. Molding.—Molding was formerly done by hand; but in modern plants machines of different types are employed, and three distinct processes are followed, each being distinguished by the condition in which the clay is used. These processes are: (1) the **soft-mud process,** which requires the clay to be very soft and pasty; (2) the **stiff-mud process,** in which the clay is used in a plastic but not soft or pasty state; and (3) the **dry-press process,** which, as its name indicates, uses dry or nearly dry clay, and forces it under great pressure into the molds.

90. The machines used for molding may be either **auger machines, plunger machines,** or combinations of these. In the auger machine, the clay is forced through a die by a series of knives arranged spirally on a horizontal shaft. The machine discharges the clay in a continuous bar, which, as it emerges from the machine, is cut by steel wires, operated by hand or automatically. In the plunger machine, the clay is forced by knives and a mud-ring downwards into a chamber, from which it passes through a die by the action of a reciprocating piston or plunger. The clay passes from the die to a cutting board, where it is cut into bricks by the same method as in the auger type.

91. Burning.—When sufficiently dried, the bricks are placed in a kiln and burned for a period ranging from 6 to 15 days, according to the type of the kiln and the kind of fuel used. The bricks made by the dry-press process do not require drying, and are removed from the machine directly to the kiln. Where improved methods of burning are in use, the old style of kiln, built of green bricks and plastered with clay on the outer surface to retain the heat, is discarded, and stationary kilns with permanent brick walls are employed.

Kilns operated by the down-draft method are usually preferred for burning brick made by the dry-press process. A full description of these methods and of the kilns used is beyond the scope of this Course; a few general features only will be given here.

92. The kilns used in both the up-draft and the down-draft method consist of permanent brick walls 2 or more feet thick and about 14 feet high, covered with a tight arched roof of brick. They are built in two forms: the rectangular and the circular, or beehive. The dimensions vary according to the capacity desired: a common size for the rectangular is 20 ft. $\times$ 80 ft.; and for the circular, a diameter of 18 or 20 feet. To prevent rupture through the expansion caused by the heat, the circular form is provided with steel bands or hoops about 4 inches wide, and the rectangular form has either steel or wooden posts with steel tie-rods. In both methods, the kilns are constructed so that they may be operated on the principle of utilization of the waste heat contained in the burned bricks during cooling. This is accomplished by dividing the kiln into compartments, some of which contain bricks in process of burning, others of which contain bricks in a state of cooling, and still others of which are being charged. The kilns operated by this method are called **continuous.** Their form is generally rectangular, and they are provided with one very high chimney, instead of the several low ones used in the intermittent types. The bricks are piled in sections. The partitions between the sections are usually made of paper. The fire is started in one section, and the heated gases, after passing through the bricks in that section, are drawn through the bricks in the next section, so as to dry them before starting the fire in that section. After the bricks are dried in this manner, the fire is started and the process repeated until the bricks in all the sections have been burned. The number of sections in a kiln varies from four to twenty, depending on the size of the kiln.

The temperature required to complete the burning is from 2,200° to 2,600° F. Where the old-style kilns are in use, the

common bricks are divided into three classes; namely, **arch**, **red**, and **salmon** or **pale**; they are all found in the same burning. The arch bricks are those used to form the arches or fireplaces, and from their contact with the fire are overburned, hard, brittle, and weak. The red bricks are the properly burned bricks from the interior of the kiln, and are the best quality. The salmon bricks are those from the exterior of the kiln; they are underburned, and too soft for use in important structures. With the improved types of kilns, the bricks are divided into but two classes: **hard** bricks, which are the properly burned bricks, and **soft** bricks, which are the salmon or underburned bricks. In some localities, these classes are designated as **firsts** and **seconds**, respectively.

93. Rank of Bricks.—Authorities differ as to the relative merits of bricks made by the different methods. It is, however, generally considered that, in regard to regularity of form, pressed brick ranks first, stiff-mud brick next, and soft-mud brick last; that in compactness and uniformity of texture, hand-molded bricks rank first, soft-mud bricks second, stiff-mud bricks third, and dry-clay bricks last. In regard to strength, stiff-mud bricks are stronger than hand-made bricks from the same clay.

ENAMELED, GLAZED, AND FIREBRICK

94. Enameled and Glazed Brick.—The terms "enameled" and "glazed" are used to designate bricks that have one surface or one surface and one end glazed. Enameled bricks are made from clay containing a large percentage of fireclay. The enamel of porcelain may be applied to the bricks either before or after they are burned. The application of the enamel before burning produces the best quality, because, in burning, the enamel fuses and unites with the body of the brick and therefore does not chip or peel off. The color is usually white or light yellow.

Glazed brick is made by applying to the unburned brick, on the side that is to be glazed, a slip composed of kaolin, flint, and feldspar; the slip adheres to the clay and holds the

glaze. The glaze is composed of materials that fuse readily and render the surface vitreous; pigments of metallic oxides are added to the glaze to produce any desired color.

A glazed brick is distinguished from an enameled brick by chipping off a piece of the brick. The glazed brick will show a well-defined line of demarcation between the glaze and the body of the brick, while the enameled brick will not show such a line.

95. Firebrick.—Firebricks are made from a refractory clay, called **fireclay**, by similar processes to those used for ordinary bricks. The clay is excavated, ground, tempered, and molded, and then burned in kilns at a heat that is slowly increased until it attains a very high temperature. The bricks are then allowed to cool gradually.

Fireclay may be defined as a natural combination of hydrated silicate of alumina with silica and alumina in various states of subdivision, and sufficiently free from alkaline silicates, and from iron and lime, to resist vitrification at high temperatures. The sulphide of iron (pyrites), oxide of iron, lime, soda, potash, and magnesia are injurious, and their presence to the extent of 3 per cent. should cause rejection of the brick.

A properly burned firebrick should be of a uniform color —white or white mixed with brown throughout. Dark central patches with concentric rings of various colors indicate that the brick was burned too rapidly.

In laying firebrick, the best results are secured by laying the bricks in a thin paste composed of the same clay from which they were manufactured. The bricks should be dipped in water as they are used, and the joints should be as thin as possible.

OTHER KINDS OF BRICK

96. Lime-Sand Brick.—Lime-sand bricks, as the name indicates, are manufactured from a mixture of lime and sand; the proportions used are from four to six parts of lime and ninety-four to ninety-six parts of sand. The lime is reduced to a fine powder in a pulverizing mill, and

mixed with the sand; it is then brought to a plastic condition by the addition of water. The mixture is pressed into molds, and the molded bricks are placed in a metal cylinder that holds, according to its dimensions, from 10,000 to 20,000 bricks. The cylinder is hermetically sealed, and the bricks are heated by steam for about 10 hours, during which time the heat causes the hydrated lime and silicic acid of the sand to combine and form the silicate of lime that gives the brick its hardness.

97. Cement-sand bricks are manufactured in a similar manner to the lime-sand brick, Portland cement being used in place of the lime.

PROPERTIES OF BRICK

98. Size and Weight.—The dimensions of bricks vary considerably. The standard adopted by the National Brickmaker's Association is, for common clay brick, $8\frac{1}{4}$ in. $\times$ 4 in. $\times$ $2\frac{1}{4}$ in., and for face or pressed brick (clay) $8\frac{3}{8}$ in. $\times$ $4\frac{1}{8}$ in. $\times$ $2\frac{1}{4}$ in. The weight of a common clay brick is about

TABLE V

WEIGHT AND STRENGTH OF BRICK

Kind of Brick	Weight Pounds Per Cubic Foot	Crushing Strength Pounds Per Square Inch
Best pressed-clay	150	5,000 to 15,000
Common hard-clay	125	5,000 to 8,000
Soft-clay	100	450 to 600
Lime-sand	120	3,600 to 7,600
Firebrick	120	1,000 to 1,500

$4\frac{1}{2}$ pounds; that of a pressed-clay, enameled brick, about 7 pounds. Enameled and glazed bricks are made in two sizes: English size, 9 in. $\times$ 3 in. $\times$ $4\frac{1}{2}$ in.; American size, $8\frac{3}{8}$ in. $\times$ $2\frac{1}{4}$ in. $\times$ $4\frac{1}{8}$ in. The usual dimensions for firebricks are 9 in. $\times$ $4\frac{1}{2}$ in. $\times$ $2\frac{1}{2}$ in.; various sizes and forms are made

to suit the required work. The dimensions of the lime-sand bricks are $8\frac{3}{8}$ in. × $4\frac{1}{8}$ in. × $2\frac{15}{16}$ in. The weight varies between 5 and 6 pounds.

Table V gives the approximate weight and resistance to crushing of brick.

99. Requisites for Good Brick.—Bricks of good quality should be of regular shape, with parallel surfaces, plane faces, and sharp square edges. They should be of uniform texture, and should be burnt hard. They should be thoroughly sound, free from cracks and flaws, and should emit a clear ringing sound when struck a sharp blow. A hard well-burned brick should not absorb more than one-tenth of its weight of water; it should have a specific gravity of 2 or more. The crushing strength of a brick laid flat should be at least 6,000 pounds per square inch. The modulus of rupture should be at least 1,000 pounds per square inch.

100. Testing Brick.—Brick are tested for absorption, resistance to freezing, strength, specific gravity, etc. in the same manner and by the same methods described for the testing of stone.

CEMENTING MATERIALS AND MORTAR

LIMES

1. Introduction.—The materials employed for cementing or binding together the stones of structures are produced by the calcination of calcareous minerals. As these minerals differ in their composition, they impart different properties to the calcined product. These properties furnish a basis for classifying the cementing materials, which are divided into the following three great classes: *common lime*, *hydraulic lime*, and *hydraulic cement*.

2. Common lime, commercially called **quicklime,** is manufactured by calcining, or burning, at a temperature of from 1,400° to 2,000° F., stones composed of pure or very nearly pure carbonate of lime. The product is practically pure oxide of calcium. It is prepared for use, converting it into calcium hydrate, by the addition of water. This process is called **slaking.**

3. Lime is slaked by spreading it on a suitable bed and moistening with water. This moistening gives rise to various phenomena: the lime almost immediately cracks, swells, and falls into a fine white powder, with a hissing sound and the evolution of heat and steam; the volume is increased from 2 to $3\frac{1}{2}$ times the original bulk.

The same process takes place slowly by the absorption of moisture from the atmosphere: the lime falls into powder with increase of volume, but without perceptible heating. Lime slaked in the latter way is said to be **air-slaked;** it is

deficient in setting properties, and should not be employed for structural purposes.

The amount of water required for slaking is about one-third the volume of the lime. The entire amount should be applied by sprinkling at one operation. The addition of cold water after the slaking has commenced depresses the temperature and renders the lime granular and lumpy. An excess of water reduces the binding qualities.

4. The quality of lime is indicated by the readiness with which the lumps fall to powder during slaking. Good lime should be free from unslaked lumps, the presence of which indicates that the limestone was not pure or that the process of calcination was imperfect.

5. The common limes will not harden under water or in damp places excluded from contact with the air. In the air, they harden by the gradual formation of carbonate of lime, due to the absorption of carbonic acid. In hardening, the lime paste shrinks to such an extent that it cannot be employed for mortar without a large percentage of sand. Slaked lime mixed into a paste may be kept for an indefinite time without deterioration, if protected from contact with the air. This is accomplished by covering the lime with the sand that is to be subsequently incorporated with it in making the mortar for which the lime is intended.

6. **Hydrated lime** (calcium hydrate) is manufactured by crushing and grinding lump lime to a fine powder, and hydrating it by sprinkling it with water. This operation is performed in a shallow pan furnished with plows, which turn the lime over so that all parts are thoroughly wetted. The heat evolved drives off the surplus water, leaving the hydrated lime in the form of a fine powder.

Hydrated lime is considered better for use with Portland cement than ordinary slaked lime, because it is more thoroughly slaked and is easily handled and measured.

7. **Hydraulic Lime.**—The name **hydraulic lime** is given to a lime that possesses the ability to harden under

water without the access of air. It is produced by burning limestone containing a small amount of clay. It is very extensively used in Europe, under the name of *lime of Teil.* In the United States, it is but little used, as natural cement takes its place.

HYDRAULIC CEMENTS

CLASSIFICATION AND GENERAL PROPERTIES

8. The hydraulic cements are divided into three main classes; namely, *Portland cement*, *natural cement*, and *pozzuolana.* These cements differ from the limes by not slaking after calcination. They can be formed into paste with water, without any sensible increase in volume and with little if any disengagement of heat, except in certain instances among those varieties that contain the maximum amount of lime. As they do not shrink in hardening, they can be used with or without sand; but for the sake of economy, sand is used.

The *activity*, or *time of setting*, of hydraulic cements is variable; some set under water at 65° F. in 3 or 4 minutes; others require as many hours. The point at which the setting is considered to begin is when the stiffening of the mass first becomes perceptible, and the end of the setting is when cohesion extends through the mass sufficiently to offer such resistance to any change of form as to cause rupture before any appreciable deformation takes place.

The binding and economic value of a cement is affected by the degree of fineness to which it is ground. Coarse particles have no setting power, and act as an adulterant.

PORTLAND CEMENT

9. **Portland cement** is produced by pulverizing the clinker obtained by burning to semifusion an intimate artificial mixture of finely ground carbonate of lime, silica, alumina, and iron oxide in definite proportions.

10. Requisites of Good Portland Cement.—Good Portland cement must possess the following characteristics:

1. *Color:* dull bluish gray; any variation from this indicates some impurity.

2. *Fineness:* it must leave by weight a residue of not more than 8 per cent. on a No.-100 sieve, and 25 per cent. on a No.-200 sieve.

3. *Time of setting:* it must develop the initial set in not less than 30 minutes, and the hard set in not less than 1 hour nor more than 10 hours.

4. *Specific gravity:* when thoroughly dried at 212° F., the specific gravity must be not less than 3.10.

5. *Tensile strength:* briquets 1 inch square in section must show the following minimum tensile strengths in pounds per square inch: neat cement, 24 hours in moist air, 150 to 200; 7 days, 450 to 550; 28 days, 550 to 650: mortar composed of one part cement and three parts sand, 7 days, 150 to 200; 28 days, 200 to 300.

6. *Soundness:* pats of neat cement about 3 inches in diameter and $\frac{1}{2}$ inch thick at the center and tapering to a thin edge, kept (*a*) in moist air for 24 hours, (*b*) in air at normal temperature for 28 days, (*c*) in water at 70° F. for 28 days, and (*d*) in an atmosphere of steam above boiling water for 5 hours, must remain firm and hard and show no signs of distortion, checking, cracking, or disintegration.

11. Weight.—The weight of 1 cubic foot of Portland cement, packed, ranges from 100 to 120 pounds. A cubic foot, loose, averages about 92 pounds. A barrel weighs, net, about 376 pounds, and the average weight of the barrel is 20 pounds. The number of cubic feet of packed cement contained in a barrel varies between $3\frac{1}{2}$ and 4 cubic feet. Measured loose, the volume ranges from $3\frac{3}{4}$ to $4\frac{1}{2}$ cubic feet.

NATURAL CEMENT

12. The **natural cements** are made by calcining natural argillaceous or silicious limestones at a heat just below fusion and grinding the product to powder. On account of the

difference in the rocks from which they are made, natural cements are extremely variable. They are usually known by the name of the place where they are made, as *Rosendale*, *Louisville*, *Utica*, *Akron*, etc.

13. Requisites for Good Natural Cement.—Good natural cement must possess the following characteristics:

1. *Color:* brown.

2. *Fineness:* it must leave by weight a residue of not more than 10 per cent. on a No.-100 sieve, and not more than 30 per cent. on a No.-200 sieve.

3. *Time of setting:* it must develop the initial set in not less than 10 minutes, and the hard set in not less than 30 minutes, nor more than 3 hours.

4. *Specific gravity:* when thoroughly dried at 212° F., the specific gravity must not be less than 2.8.

5. *Tensile strength:* briquets 1 inch square in section must show the following minimum tensile strengths in pounds per square inch: neat cement, 24 hours in moist air, 50; 7 days, 100; 28 days, 200: mortar one part cement and three parts sand, 7 days, 25; 28 days, 75.

6. *Soundness:* pats of neat cement about 3 inches in diameter and ½ inch thick at the center, and tapering to a thin edge, kept (*a*) in moist air for 24 hours, (*b*) in air at normal temperature for 28 days, (*c*) in water at 70° F. for 28 days, must remain firm and hard and show no signs of distortion, checking, cracking, or disintegration.

14. Weight.—The average weight of natural cement, packed, is 74 pounds per cubic foot. A barrel weighs 265 pounds, net, and contains approximately 3.58 cubic feet of packed cement, or 4.77 cubic feet of loose cement.

POZZUOLANA

15. Description and Properties.—The name **pozzuolana** is given to cement made from a mixture of definite proportions of blast-furnace slag and slaked lime. This class of cement is characterized by its light lilac color and its low

specific gravity (2.6 to 2.8). When properly made, it contains no free or anhydrous lime, does not warp nor swell, but is liable to fail from cracking and shrinking (at the surface only) in dry air. Mortars and concretes made from pozzuolana cements approximate in strength similar mixtures of Portland cement, but their resistance to cracking is less.

Pozzuolana cement requires but little water to cause induration; it may therefore be employed in comparatively dry mixtures, but should be well rammed. For permanency or durability, it requires the presence of constant or continuous moisture; consequently, it is well adapted for use in situations continually exposed to moisture, as in foundations, sewers, and underground work generally; it is also well adapted for use in structures exposed to the action of sea-water. It does not resist blows, attrition, or mechanical wear, and so it should not be employed in situations where it would be subjected to the action of these agents.

16. Requisites for Good Pozzuolana.—The requisites of good pozzuolana are as follows:

1. Ninety-seven per cent. must pass through a sieve having 10,000 meshes per square inch.

2. When thoroughly dried at 212° F., its specific gravity must be between 2.7 and 2.8.

3. Pats of neat cement kept under wet cloths until set and then placed and kept in fresh water for 28 days, must show no distortion or cracks.

4. It must not acquire its initial set in less than 45 minutes, and must acquire its final set in 10 hours.

5. Briquets made of neat cement, after being kept in air under a wet cloth for 24 hours and the rest of the time in fresh water, must show a tensile strength per square inch of 350 pounds at the end of 7 days, and 500 pounds at the end of 28 days. Mortar composed of one part cement and three parts sand must show, at 7 days, 140 pounds, and at 28 days, 220 pounds per square inch.

17. Weight.—The average weight of pozzuolana per cubic foot, loose, is $67\frac{1}{2}$ pounds. A barrel weighs about 330 pounds net, and contains about 4.96 cubic feet of loose cement.

SILICA CEMENT

18. Silica cement is the name given to a mixture of Portland cement and sand ground together until extremely fine. The sand is an adulteration, but owing to the extreme fineness, the cementing capacity is increased and thus to a certain degree offsets the effects of the sand adulteration; so that, when silica cement is made of equal parts of cement and sand, its tensile strength approximates that of the neat cement. It is claimed that silica cement possesses the peculiarity of making a mortar that will resist the action of the elements and is impermeable to moisture.

TESTING CEMENT

19. Character of Tests.—The quality or constructive value of a cement is ascertained by submitting a sample of the cement to a series of tests. The properties usually examined are *specific gravity*, *activity*, *soundness*, *fineness*, and *tensile strength*. Chemical analysis is sometimes made to detect adulterations and to ascertain if harmful constituents are present in inadmissible quantities. As the tests cannot be made at the site of the work, it is usual to sample each lot of cement as it is delivered, and send the samples to a testing laboratory.

20. Sampling.—The cement is sampled by taking a small quantity (1 to 2 pounds) from the center of the package. The number of packages sampled in any given lot depends on the character of the work, and varies from every package to one in five or one in ten. When the cement is brought in barrels, the sample is obtained by boring with an auger either into the head or into the center of the barrel, drawing out a sample, and then closing the hole with a piece of tin

tacked firmly over it. For drawing out the sample, a brass tube sufficiently long to reach the bottom of the barrel is used. The tube is thrust into the barrel, turned around, and pulled out; the core of cement is then knocked out into the sample can, which is usually a tin box with a tightly fitting cover. When the cement is in bags, the sample is taken from the surface to the center. Each sample should be labeled, stating the number of the sample, the number of bags or barrels it represents, the brand of the cement, the purpose for which it is to be used, and the date of delivery and that of sampling. The sample should be sent at once to the testing office, and none of the cement should be used until the report of the tests is received. The testing of cement ordinarily consumes 30 days; therefore, the supply must be gauged so that a sufficient quantity will be kept on hand to allow the tests to be made without delay to the work of construction.

METHODS OF TESTING

21. Test for Specific Gravity.—The specific gravity test is used to determine whether the cement is thoroughly calcined and whether it is adulterated (adulteration and underburning reduce the specific gravity; overburning increases it). The specific gravity is most conveniently determined by a **specific-gravity bottle,** such as is shown in Fig. 1. This consists of a glass flask having two bulbs, the lower somewhat exceeding the upper in capacity. The exact capacity of the lower bulb is of no importance. On the neck between the bulbs is a mark *a*; on the neck of the upper bulb is a similar mark *b*. The capacity of the space between the marks *a* and *b* is accurately determined. The volume of that space is usually either 500 or 1,000 cubic centimeters. In ascertaining the specific gravity, the flask is filled up to the mark *a* with some liquid that will not act on the cement (as turpentine or benzine), and weighed. Cement previously dried at 212° F. and cooled to a temperature of 60° F. is gradually poured into the flask until the liquid rises from *a* to *b*, and then the flask is weighed again. The excess of this

weight over the weight of the flask and liquid previously determined gives the weight of the cement. Since the weight of 1 cubic centimeter of water at 60° F. is approximately 1 gram (the exact weight is .999 gram), the volume, in cubic centimeters, of the liquid displaced is numerically equal to the weight, in grams, of an equal volume of water. Therefore, the specific gravity of the cement is equal to its weight, in grams, divided by the volume, in cubic centimeters, of the liquid displaced. If the weight of the cement, in grams, is represented by W and the volume of the part $a\,b$ of the flask is 1,000 cubic centimeters, then Sp. Gr. = .001 W.

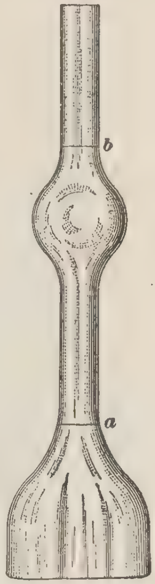


FIG. 1

22. Test for Fineness.—The **fineness** is determined by measuring the percentage that will not pass through sieves having a certain number of meshes per linear or per square inch. Two circular sieves having a diameter of 7.87 inches and a depth of 2.36 inches are generally used, one known as No. 100 and having 96 to 100 meshes to the linear inch, or 10,000 per square inch; and the other known as No. 200, having from 188 to 200 meshes to the linear inch, or 40,000 per square inch. The test is conducted as follows: 100 grams (3.52 ounces) of the cement is dried at a temperature of 212° F., then placed in the No. 200 sieve, which, with its pan and cover attached, is shaken by hand at the rate of about 200 movements per minute. After 1 minute of continuous shaking, the residue remaining on the sieve is weighed, then placed on the No. 100 sieve, and the operation repeated. The results are reported to the nearest tenth of 1 per cent.

23. Test for Activity.—The **activity, or time of setting**, is ascertained by submitting pats of cement paste to the penetrating effect of weighted needles known as **Gilmore's** and **Vicat's needles.** To make the test, prepare a

plastic mortar by mixing the cement with from 25 to 30 per cent. of its weight of clean water having a temperature between 65° and 70° F.; then make a cake or pat 2 or 3 inches in diameter and $\frac{1}{2}$ inch thick, immerse it in water having a temperature of 65° F., note the time required for the cement to set hard enough to bear respectively a Gilmore's needle made of steel wire $\frac{1}{12}$ inch in diameter and loaded to weigh $\frac{1}{4}$ pound, and a needle $\frac{1}{24}$ inch in diameter loaded to weigh 1 pound. In use, the points of the needles rest on the cement, and are held between the fingers in a vertical position. The moment when the coarse needle fails to sink into the cement is called the time of initial setting; when the fine needle no longer penetrates, the moment of final setting.

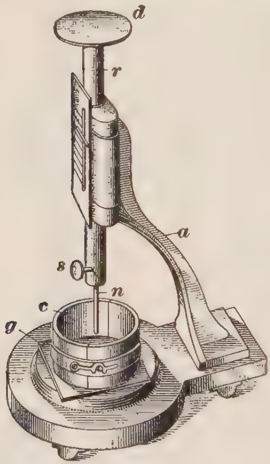


FIG. 2

The **Vicat needle,** shown in Fig. 2, consists of a frame *a*, carrying a movable rod *r*, which has a cap *d* at one end, and a steel-wire needle *n*, 1 millimeter (.039 inch) in diameter, at the other. The cap, rod, and needle weigh 300 grams (10.5802 ounces). The rod, which can be held in any desired position by the screw *s*, carries an indicator that moves over a scale graduated in centimeters and attached to the frame. The pat of cement to be tested is placed in a rubber ring *c*, resting on a glass or metal plate *g*; the needle is brought into contact with its surface and released. The setting is said to have commenced when the needle ceases to pass a point 5 millimeters (.2 inch) above the upper surface of the glass plate, and is said to have terminated the moment the needle does not sink visibly into the mass.

24. Tests for Soundness.—The tests for **soundness** have for their object the determination of those properties

that tend to impair the strength and durability of a cement. They are divided into two classes: (1) **normal tests,** made in the air or water at a temperature of about 70° F., and (2) **accelerated tests,** made in air, steam, or water at a temperature of 115° F. or more. Failure is revealed by a cracking, checking, swelling, or disintegration, or a combination of all these phenomena. A cement that remains perfectly sound is said to be of **constant volume.**

The tests for soundness are carried out by making two pats of pure-cement paste, 3 or 4 inches in diameter and about ½ inch thick at the center, tapering to a thin edge. Each pat is placed on a piece of glass and covered with a damp cloth, or placed in a closet protected from currents of dry air. At the end of 24 hours, one pat still attached to the glass is taken for the normal test and is immersed in water, where it is left for 28 days; it is examined from day to day to see if it shows any cracks or distortion. For the accelerated test, the other pat is placed in water at a temperature of about 70° F., and supported in a rack above the bottom of the receptacle. The temperature is raised gradually to the boiling point. This temperature is maintained for 6 hours; then the pat is allowed to cool, and is examined for evidence of cracks or distortion.

25. Test for Strength.—The strength of cement is determined by submitting a specimen, called a **briquet,** of known cross-section to a tensile stress. The reason for adopting tensile tests is that comparatively light stresses produce rupture, and that, since mortar is less strong in tension than in compression, it fails in most cases by tensile stress, even though the masonry is under compression. The briquets are tested at two periods: the first, 7 days, and the second, 28 days after molding. In each case the briquets remain for the first 24 hours in moist air, and the rest of the time in fresh water at 70° F. A cement giving an extremely high strength at the 7-day period should be regarded with suspicion, and the tests for soundness should be made with great care.

26. The method employed to ascertain the tensile strength is to make two tests, one with neat or pure cement, and one with a mixture of cement and sand:

1. A certain quantity of pure cement is placed on a glass plate and mixed with just sufficient water to make it plastic. The paste is then pressed firmly into brass molds. The shapes of these molds vary, the form shown in Fig. 3 being a very common one. The form recommended in 1905 by the American Society for Testing Materials is similar to that shown in Fig. 3, with the corners *a* rounded off. After the mold is filled, it is covered with a damp cloth, or is placed in a moist closet and allowed to stand for 24 hours. At the end of this period, the block or briquet of cement is removed from the mold and immersed in water, where it remains until the time for testing arrives.

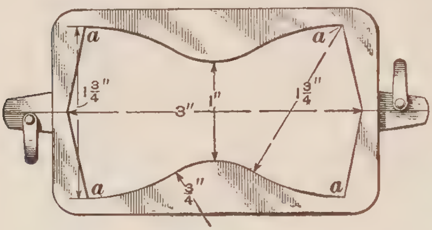


FIG. 3

2. One part of cement and two or more parts of sand are carefully measured by weighing, and then thoroughly and intimately mixed dry on a plate of glass or other non-absorbing surface. The mixture is then drawn together in the form of a mound, and a depression made in its center, into which just sufficient water to produce the desired consistency is poured. The material on the outer edge is turned into the depression, and the whole mass is thoroughly mixed for about 1 minute. The mortar so made is immediately placed in the mold and compacted with a moderate pressure of the fingers; the filled mold is then covered with a moist cloth or is placed in a moist closet and allowed to stand for 24 hours, after which it is removed from the mold and immersed in water, where it remains until tested.

In order to compare different brands of cement, it is essential that the sand used for the mortar be of exactly uniform composition and quality as regards size, sharpness,

surface of grains, and degree of dampness. As it is difficult to obtain sand of so uniform a quality, a standard sand made by crushing quartz is used. This standard sand is of such fineness that it will all pass a No.-20 sieve (400 meshes to the square inch) and be caught on a No.-30 sieve (900 meshes to the square inch). On important works, the tests are usually made with the actual sand that is to be used.

27. When the briquets have obtained the age of 7 and 28 days, respectively, they are removed from the water and

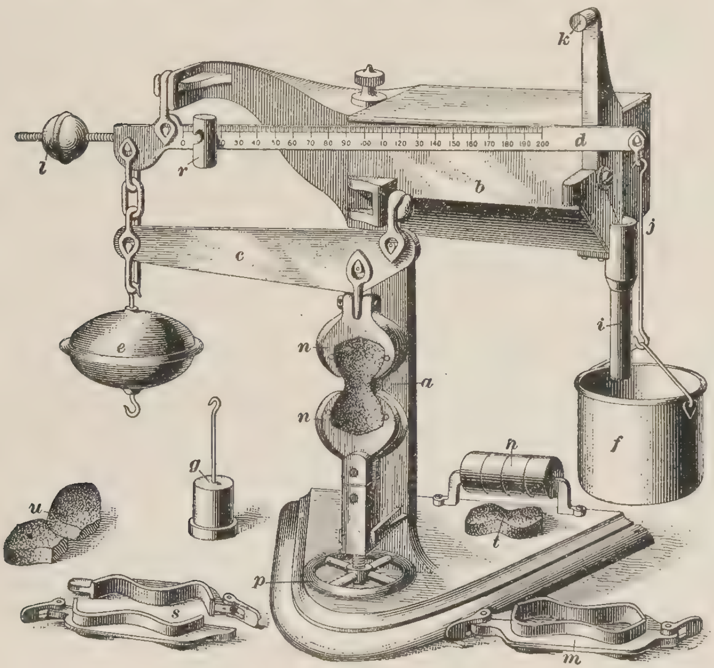


FIG. 4

subjected to a tensile stress until rupture takes place. For this purpose, several machines are used. The one most commonly employed is the **shot machine**, shown in Fig. 4. To make a test, hang the cup *f* on the end of the beam *d*, set the poise *r* at the zero mark, and balance the beam by turning the ball *l*, which screws in and out. Fill the hopper *b*

with fine shot, place the briquet in the clamps *n*, *n*, and adjust the hand wheel *p* so that the graduated beam will rise nearly to the stop *k*. Open the automatic valve *j* so as to allow the shot to run slowly into the cup *f*. When sufficient shot has passed into the cup to break the briquets, the beam *d* will drop and close the valve *j*. When this occurs, remove the cup *f* with the shot in it and hang the counterpoise weight *g* in its place. Hang the cup *f* on the hook under the ball *e*, and weigh the shot, using the poise *r* on the beam *d*, which is graduated in pounds and fractions, and the weights *h* on the counterpoise *g*. The force required to break the briquet is then found by multiplying the weight of the shot by the constant of the machine.

In the figure, *s* shows the two halves of a mold; *m*, the mold ready to receive the plastic cement; *t*, the hardened briquet before testing; and *u*, the briquet broken in the machine.

28. Rough Tests.—As it is frequently necessary to use cement before the laboratory tests can be completed, as well as to ascertain if the unsampled barrels are running the same as the sampled ones, it is essential that rapid or rough tests be made at the work. These may be performed as follows: The setting and hardening is determined by noting the time that elapses until pats of the cement and the mortar resist penetration under a light pressure of the thumbnail. The soundness is ascertained by placing a pat of the cement in water and raising the temperature to the boiling point; if the pat shows no signs of checking or cracking, the cement may be considered sound.

SAND

29. Sand is an aggregation of loose, incoherent grains of crystalline structure, derived from the disintegration of rocks. It is called **silicious, argillaceous,** or **calcareous,** according to the character of the rock from which it is derived. It is obtained from the seashore, the banks and beds of rivers, and land deposits. The first class, called **sea sand,** contains alkaline salts that attract and retain moisture and cause efflorescence in brick masonry. The second, termed **river sand,** is generally composed of rounded particles, and may or may not contain clay or other impurities. The third, called **pit sand,** is usually composed of angular grains, and often contains clay and organic matter; when washed and screened, it furnishes a good sand for general purposes.

30. Uses of Sand.—The principal use of sand is in making mortar, to prevent excessive shrinkage and reduce the amount of lime or cement used. Lime adheres better to the particles of sand than to its own particles; hence, sand is considered as adding strength to lime mortar. On cement mortar, on the contrary, it has a weakening effect. Sand is also used as a cushion to distribute the pressure of structures over soft soils, as a foundation and joint filling for pavements, and for plastering.

31. Properties and Testing of Sand.—Dry sand weighs from 80 to 115 pounds per cubic foot. Moist sand occupies more space and weighs less per cubic foot than dry sand.

The voids of ordinary sand range from one-fourth to one-half of the volume. The more uneven the grains in size, the smaller the percentage of voids.

The fineness of sand is measured by determining the percentage passing through five sieves, the first having

400 meshes, the second 900, the third 2,500, the fourth 6,400, and the fifth 28,900 per square inch. When the grains range from $\frac{1}{16}$ to $\frac{1}{8}$ inch, the sand is called **coarse**; when from $\frac{1}{16}$ to $\frac{1}{24}$ inch, **fine**; and when from $\frac{1}{30}$ to $\frac{1}{60}$ inch, **very fine**; when it is composed of sizes varying within these limits it is termed **mixed sand.**

The cleanness of sand may be ascertained by rubbing a small amount in the palm of the hand and after throwing it out, noticing the amount of dirt left. The sharpness may be determined by examining the sand with a magnifying glass.

The presence of salt may be ascertained by placing a small portion of the sand in a clean bottle with distilled water. After shaking, it is allowed to settle; a few drops of pure nitric acid are added, and then a few drops of a solution of nitrate of silver. A white precipitate indicates the presence of salt; a faint cloudiness may be disregarded.

The presence of clay is ascertained by shaking a small quantity of sand in a glass of clear water, then allowing it to stand and settle for a few hours. The sand and clay will separate into two well-defined layers.

32. Sand is prepared for use by (*a*) screening to remove the pebbles and coarser grains, the fineness of the meshes of the screen depending on the kind of work in which the sand is to be used; (*b*) washing to remove salt, clay, and other foreign matter; (*c*) drying, if necessary. When dry sand is required, it is obtained by evaporating the moisture either in a machine, called a **sand dryer,** or in large, shallow iron pans supported on stones, with a wood fire placed underneath.

MORTAR

33. Mortars for structural purposes are composed of lime or cement and sand, mixed to the proper consistency with water. The proportions of the ingredients depend on the character of the work in which the mortar is to be used. The quality of the mortar depends on the quality of its constituents, the proportions in which they are used, and the method by which they are mixed.

34. The sand used for mortar must be clean, that is, free from dirt and organic matter and an excess of clay (5 per cent. of clay is considered detrimental). The fineness will depend on the character of the work for which the sand is to be used: if in rubble masonry and concrete, it should be a mixture of coarse and fine grains, the coarse predominating; if in ashlar or brick masonry, a fine sand should be used. The best sand is that in which the grains are of different sizes; the more uneven the sizes, the smaller will be the amount of voids, and hence the less the amount of cement required. In the absence of suitable sand, stone dust, pulverized slag, brick, and burnt clay may be substituted without apparently affecting the strength or durability of the mortar.

35. The water should be fresh and clean, and free from mud and vegetable matter. Salt water may be used, but it retards the setting. The quantity of water should be just sufficient to produce a plastic consistency suitable for the work for which the mortar is intended. The consistency of mortar for masonry should be such that it will stand in a pile and not be fluid enough to flow.

36. Cement mortars should be used before they take the initial set. Only as much mortar should be mixed as can be utilized before the initial set occurs. The adding of water and remixing of cement that has once set should not be

permitted, because the resulting mortar will be weak and friable.

The setting of Portland cement is retarded by cold; but the ultimate strength appears to be only slightly affected by freezing. The natural cements are generally completely ruined by freezing, and therefore should not be used at low temperatures. In making cement mortars at temperatures below 30° F., it is customary to either heat the ingredients, or add salt to the water. The amount of salt required is considered to be about 10 per cent. of the weight of water used. Other substances, such as sugar, glycerine, alcohol, gypsum, etc., are sometimes mixed with mortar to hasten or retard the setting and to increase strength or hydraulicity, but they are of doubtful effectiveness, and tend to lower the strength and durability of the mortar.

37. Mixing the Ingredients.—In mixing mortar, it is usual to designate the amount of each ingredient by a ratio, such as 1 to 1, 1 to 2, 1 to 3, etc., which signifies that the mortar is composed of one part of lime or cement to one, two, or three parts of sand, respectively. The first number of the ratio always indicates the amount of cement, which for convenience is taken as unity. The proportion of the ingredients varies according to the work for which the mortar is used. The unit for measuring these ingredients is usually the commercial barrel of cement. The cement is sometimes shipped in bags, but as a certain number of bags, usually three for natural and four for Portland cement, represents a barrel, the barrel still forms the most convenient unit. In specifying the barrel as the unit, it must be clearly stated whether it is the volume of packed or loose cement that is intended, because packed cement when emptied from the barrel increases in volume to such an extent that a nominal 1-to-3 mortar is changed to an actual 1-to-4.

38. The ingredients of mortar are usually mixed by hand. Sometimes, on extensive works where large quantities can be used within a short time, mechanical mixers are employed with the advantage of lessening the labor and insuring good

work. In mixing by hand, a platform or box should be employed. The sand and cement should be spread in layers, with a layer of sand equal to about one-half that to be used in the batch, spread evenly as a bed; the dry cement should be spread over this layer, and the remainder of the sand spread over the cement. The layers of sand and cement should then be turned and mixed with shovels, until a thorough incorporation is effected. The dry mixture should then be spread out, a bowl-like depression formed in the center, and all the water intended to be used poured into it. The dry material from the outside of the basin should be thrown in until the water is taken up, and then worked with shovel and hoe until it assumes a plastic condition. Or, the dry mixture may be shoveled to one end of the box, and the water poured in at the other end. The mixture is then drawn into the water with a hoe, a small quantity at a time, and mixed with the water until a plastic mass is formed.

39. Lime mortar is ordinarily composed of one part of slaked lime to four parts of sand. This kind of mortar should not be used in foundation work, below the water-line, or in continually damp situations; neither should it be used in freezing weather.

40. Portland-cement mortar is composed of Portland cement and sand in proportions that vary from one part of cement and one part of sand to one part of cement and six parts of sand, this variation being due to the strength of the mortar desired. The common proportion for ordinary masonry is one part of cement to three parts of sand. For pointing face joints, one part of cement to either one or two parts of sand is used.

41. Natural-cement mortar is usually composed of one part of cement and two parts of sand. This proportion is found to possess sufficient adhesion and resistance to crushing for ordinary masonry above ground.

42. Cement-Lime Mortar.—In the construction of brick walls, faced with pressed brick, it is necessary to have

a plastic, slow-setting mortar, in order that the mason can do neat work. This quality of mortar is obtained by using a mixture of one-fourth natural cement, one-fourth lime, and one-half sand.

It is sometimes necessary to use a mortar that will set more quickly than Portland cement, and ultimately attain a greater strength than either natural cement or lime. This result may be obtained by mixing lime paste with Portland cement or by using a mixture of equal parts of natural and Portland cement; either mixture will set rapidly and will attain considerable strength. The addition of the lime will increase the adhesion of the mortar to brick or stone, and also its impermeability to water.

43. Mortar Impervious to Water.—Both lime and cement mortar absorb water; consequently, they disintegrate under the action of frost. Impermeability of the mortar may be increased by carefully grading the sand and increasing the amount of cement. The addition of a small amount of lime tends to reduce the volume and number of the voids and thus aids in reducing the permeability. Practically impermeable mortar may be made by adding to the mortar a mixture of alum and soap. The proportions usually employed are $\frac{3}{4}$ pound of pulverized alum to each cubic foot of sand, and $\frac{3}{4}$ pound of potash soap to each gallon of water. The alum and soap combine and form compounds of alumina and fatty acids that are insoluble in water. The strength of the mixture is but little inferior to the strength of the mortar of the same proportions.

44. Strength of Mortar.—The strength that mortar should possess is of three kinds; namely, *compressive*, *cohesive*, and *adhesive*. The degree to which it should possess any one of these depends on the position in which it is employed. In ashlar masonry, resistance to compression is all that is required; in uncoursed rubble masonry and in brick masonry, it must possess adhesiveness, or the capacity of adhering to the surface of the stones or brick in order to prevent their displacement. In masonry of all classes that may have

to develop transverse stresses, it must possess cohesiveness or tensile strength.

The tensile and the compressive strength of a given mortar depend on the adhesive strength of the cementing medium and on the character of the aggregate. Coarse and fine sand in the proportion of about four parts of coarse grains ($\frac{8}{100}$ to $\frac{2}{10}$ inch in diameter) and one part of very fine grains (less than $\frac{2}{100}$ inch in diameter) usually produce the strongest mortar. Screenings from broken stone usually produce stronger mortars than sand, because of their greater density. Mixtures of sand and screenings often produce stronger mortar than either material alone. With the same aggregate, the strongest and most impermeable mortar is that containing the largest percentage of cement in a given volume of the mortar. With the same percentage of cement in a given volume of mortar, the strongest, and usually the most impermeable, mortar is that which has the greatest density, that is, which in a unit volume has the largest percentage of solid materials.

45. The tests that have been made to determine the various strengths of mortar show varying results, owing to the differences in the sands employed and the methods followed in conducting the tests. The resistance to compression is ascertained by submitting cubes of mortar to compressive stress in suitably arranged machines. The crushing strength of mortar is usually taken as equal to eight or ten times the tensile strength.

46. The resistance of mortar to tensile stress is measured by breaking briquets composed of cement and sand mixed in the desired proportions and tested in the manner described in Art. **27**. The tensile strength of first-class cement mortar at the end of 1 week is about as follows:

	Pounds Per Square Inch
Portland cement 1 part, sand 3 parts . .	150 to 250
Natural cement 1 part, sand 2 parts . .	120 to 150
Clear (neat) Portland cement	450 to 700
Clear (neat) natural cement	200 to 270

These values may be expected to apply to good materials. The values required by specifications are, as a rule, far below the upper limits here given. Thus, for mortar composed of one part of Portland cement and three parts of sand, tensile strengths of 100 and 150 pounds per square inch, at the end of 7 and 28 days, respectively, are required by some specifications; and for neat cement, 450 and 550, respectively, for the two ages just mentioned.

In comparing the tensile strengths of mortars, it is generally considered that a mortar composed of one part of Portland cement and four parts of sand is equal to a mortar composed of one part of natural cement and two parts of sand.

In testing the tensile strength of Portland-cement mortar, composed of one part of cement and one part of sand, it frequently happens that this mixture shows a greater strength than a paste made of cement. This result is due to the thoroughness with which the mortar is mixed and compacted in the molds.

47. The adhesive strength of a mortar depends not only on the materials of which it is composed, but also on the character and porosity of the materials to which it adheres. The experiments so far made show that the adhesion of mortar to brick surfaces is from 25 to 85 pounds per square inch, and to iron bolts from 200 to 300 pounds per square inch.

48. **Grout** is a thin or fluid mortar made in the proportion of one part of cement to one or two parts of sand. It is used to fill up the voids in walls of rubble masonry and brick. Sometimes the interior of a wall is built up dry and grout poured in to fill the voids. Unless specific instructions are received to permit its use, grout should not be employed. When used by masons without instructions, it is generally for the purpose of concealing bad work.

STONE AND BRICK MASONRY

STONE MASONRY

CLASSIFICATIONS OF MASONRY

1. Divisions of Masonry.—Masonry is divided into four classes according to the materials composing it; namely, *stone masonry*, *brick masonry*, *concrete masonry*, and *mixed masonry*. The last of these names is applied to masonry composed of stone and brick. These classes are again subdivided into several others, according to the character of construction and workmanship employed.

2. Subdivisions of Stone Masonry.—Stone masonry is subdivided into three principal classes, according to the manner in which the stone is prepared. The first class, called **rubble masonry**, is composed of rough stones as they come from the quarry. The second class, termed **squared-stone masonry**, is formed of stones roughly brought to a rectangular shape by the use of tools. The third class, called **ashlar masonry**, is composed of stones accurately cut and dressed to specified dimensions.

Each of these classes is subdivided into several others, according to the manner in which the work is executed. Combinations of the different classes are also employed under various names, and are frequently employed in a single structure.

DEFINITIONS OF TERMS USED IN STONE MASONRY

3. Parts of a Stone.—The names given to the parts of a block of stone when prepared for a structure are here defined. The **face** is the surface that will be exposed to view. The **back** is opposite the face, and parallel, or nearly so, to it. The upper and the lower horizontal surface (as laid) are called **beds**, and are distinguished, respectively, as the **top bed** and the **bottom bed**. The height or distance between the top and bottom beds is called the **rise** or **build**. The vertical sides at right angles to the face are called the **joints**. The four edges of the face, when they have been sharply defined by the use of the chisel, are termed **pitch lines** or **arrises**. A stone used at the corner of a wall and showing two faces is called **a quoin**. Small pieces of wedge-shaped stone are called **spalls**. When the blocks of stone are so large as to require machinery to move them, a hole, of the shape of an inverted truncated wedge, is cut in the center of the top bed to receive a device called a **lewis**, to which the hoisting rope from the derrick is attached. Otherwise, two holes are cut obliquely in the top bed to receive bolts with eyes for the same purpose. When the device called a **dog**, or **grab**, is used, small holes are drilled in the face and back, or preferably in the sides of the stone, to receive its points. The holes should be so placed as not to mar the appearance or affect the durability or strength of the stone, and should never be placed in the faces of fine cut stones. The holes are variously designated as **lewis holes, grab holes, dog holes, etc.**

4. Parts of a Wall.—The **footings** of a wall are the projecting courses at the base of the wall, employed for the purpose of distributing the weight over an increased area and thereby diminishing the liability to vertical settlement from compression of the ground. Footings, to have any useful effect, must be securely bonded into the body of the work and have sufficient strength to resist the cross-stresses

to which they may be subjected. If the bottom course is not solidly bedded, if any rents or vacuities are left in the beds of the masonry, or if any of the materials are unsound or badly put together, the effects of such carelessness will show themselves sooner or later and always at a period when remedial efforts are useless. Too much care cannot be bestowed on the footing courses of any structure, as on them depends much of the stability of the work.

The **face** is the portion of the wall exposed to view, and the **back** is the inner or unexposed surface.

5. Each horizontal row or layer of stone is called a **course.** Some of the courses have particular names. The **plinth,** also called the **water-table,** is a projecting course placed at or near the ground line. The **belt** and **stringing** courses are wide or narrow projecting courses placed at intervals on the face for the purpose of ornament. The course from which an arch springs is called the **springing course.** A **stretching course** is a course composed entirely of stretchers. A **header course** is a course composed of headers. A **bonding course** is a course composed of bond stones. The terms *stretcher*, *header*, and *bond* are defined further on. The **corbel course** is composed of pieces of stone projecting from the face of the wall, for the purpose of supporting a course that projects still farther. The **cornice** is the ornamental course or courses set at the top of the wall. The **blocking course** is a course of large stones set on the top of the cornice.

6. The **coping** is the finishing course at the top of the wall, and consists of large stones projecting slightly over the wall at both sides, accurately bedded on the wall and joined to one another with cement mortar. The coping is used to shelter the mortar in the interior of the wall from the weather, and to protect, by its weight, the smaller stones below it from being knocked off or picked out. Coping stones should be so shaped that water may rapidly run off them. For coping, long stones are preferable to short ones, because the number of top joints will be diminished and the

mass beneath the coping will be better protected. Additional stability is given to a coping by so connecting the stones that it will be impossible to lift one of them without at the same time lifting the ends of the two stones next to it. This is done either by means of metal **cramps,** which have the ends turned at right angles to the body of the bar, and are inserted in holes cut in the stones and fixed there with lead, or by means of dowels of wrought iron, cast iron, copper, or hard stone. The metal dowels are inferior in durability to those of hard stone, although they are superior in strength. They are fastened by pouring melted lead or sulphur around them. Copper is strong and durable, but expensive. The stone dowels are small prismatic or cylindrical blocks, each of which fits into a pair of opposite holes in the contiguous ends of a pair of coping stones and fixed with cement mortar.

The under edge of the coping should be **throated** or **dripped;** that is, grooved so that the water falling on it will not run back on the wall, but will drop from the edge.

Coping is divided into three kinds; namely, **parallel coping,** which is bedded level and is level on top; **feather-edged coping,** which is bedded level and slopes on top; **saddleback coping,** which has a curved or doubly inclined top.

7. The term **bond** is applied to the method adopted for placing the stones or bricks in a wall, by lapping them over one another, so as to prevent the vertical joints from forming a continuous straight line, the occurrence of which would produce a weak and easily separable structure. A good bond breaks the vertical joints, both in the length and in the thickness of the wall.

Various methods are employed to form the bond. The method by **headers and stretchers,** in which the vertical joints of each course alternate with the vertical joints of the courses above and below it, is the simplest and the most commonly used. In this method, shown in Fig. 1, the blocks of each course are laid alternately with their greatest

and least dimensions to the face of the wall. Those that present the longest dimension, as *a*, are termed **stretchers**; the others, as *b*, are termed **headers.** This arrangement of headers and stretchers is termed the **longitudinal bond.** Its object is to distribute the load or pressure over an increasing area downwards to the foundation. The extent to which the blocks should overlap or break joint is usually from one to one and one-half times the rise or height of the course.

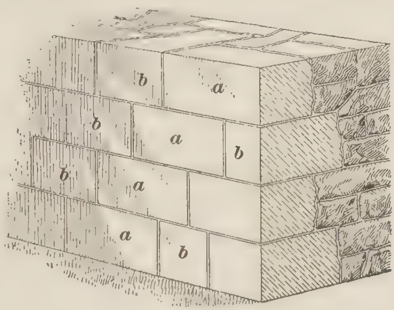


Fig. 1

The bond across the thickness of the wall is of more importance than the longitudinal bond; its object is to tie the face and the back of the wall together; this is effected by the headers. A header that extends from face to back is termed a **through bond.** When walls exceed 3 feet in thickness, it is not advisable to use through bonds, because of the liability of the long stone to become broken in the middle by unequal settlement; in its place headers that reach only a part of the distance are used. In this case, they are called **binders** (as shown at *a*, Fig. 2) and should extend from each face about two-thirds of the thickness of the wall; they should be arranged so as to cross each other alternately.

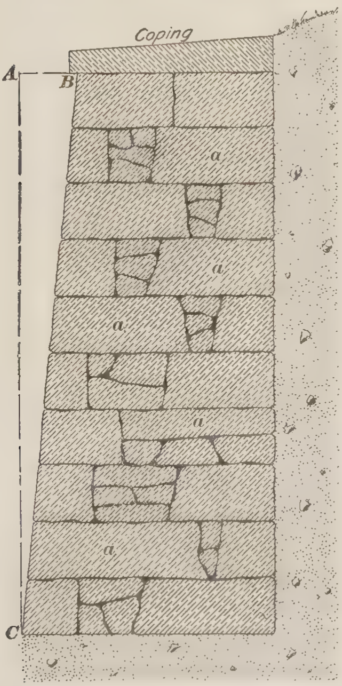


Fig. 2

The strongest bond in masonry composed of rectangular stones is that in which each course at the face of the wall contains a header and a stretcher alternately, the outer end of each header resting on the middle of a stretcher in the course below; so that rather more than one-third of the area of the face consists of ends of headers. This proportion may be deviated from when circumstances require it; but in every case it is advisable that the ends of headers should not form less than one-fourth of the whole area of the face of the wall.

8. **Joints.**—The mortar layers between the stones are called the **joints.** The horizontal joints are called **bed joints,** or simply **beds.** The end joints are called **vertical joints.** When the term *joint* is used without any qualification, a vertical joint is meant. Excessively thick joints should be avoided. For ashlar masonry, joints should be about $\frac{1}{2}$ to $\frac{1}{16}$ inch thick; for rubble masonry, they vary according to the character of the work.

Joints of masonry are finished in different ways, with the object of presenting a neat appearance and of throwing the

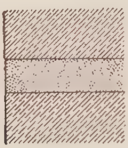


FIG. 3

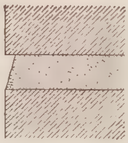


FIG. 4

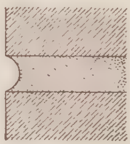


FIG. 5

rain water away from the joints. The joints most used are: (1) **Flush joints,** Fig. 3, in which the mortar is pressed flat with the trowel, and the surface of the joint is flush with the face of the wall. (2) **Struck joints,** Fig. 4, formed by pressing or striking back with the trowel the upper portion of the joint, while the mortar is moist, so as to form an outer sloping surface from the bottom of the upper course to the top of the lower course. This joint is also called **weather joint.** Masons generally form this joint so that it slopes inwards, thus leaving the upper arris of the lower course bare and exposed to the action of the weather. The reason

for forming the joint in this improper manner is that the work is more easily done. (3) **Keyed joints,** Fig. 5, formed by drawing a curved iron key or jointer along the center of the flushed joint and pressing it hard so that the mortar is driven in beyond the face of the wall. A groove of curved section is thus formed, having its surface hardened by the pressure.

9. Pointing a piece of masonry consists in scraping out the mortar in which the stone was laid, from the face of the joints for a depth of from $\frac{1}{2}$ inch to 2 inches, and filling the groove so made with clear Portland-cement mortar, or with mortar made of one part of cement and one part of sand. The necessity for pointing arises from the fact that the exposed edges of the joints are always deficient in density and hardness, and the mortar near the surface of the joint is specially subject to dislodgment, since the contraction and expansion of the masonry are likely either to separate the stone from the mortar or to crack the mortar in the joint, thus permitting the entrance of rain water, which by freezing forces the mortar from the joints.

The pointing mortar, when ready for use, should be rather incoherent and quite deficient in plasticity. Before applying the pointing, the joint must be well cleansed by scraping and brushing out the loose matter, then thoroughly saturated with water and maintained in such a condition of dampness that the stones will neither absorb water from the mortar nor impart any to it. Walls should not be allowed to dry too rapidly after pointing. Pointing should not be done either during freezing or during excessively hot weather. The pointing mortar is applied with a mason's trowel, and the joint is well calked with a calking iron and hammer. In the very best work, the surface of the mortar is rubbed smooth with a steel polishing tool. The form given to the finished joint is the same as described in Art. **8.** Pointing with colored mortar is frequently employed to improve the appearance of the work. Various colors are used, as white, black, red, brown, etc., different-colored pigments being added to

the mortar to produce the required color. Many authorities consider that pointing is not advisable for new work, as the joints so formed are not so enduring as those finished at the time the masonry is built. Pointing is, moreover, often resorted to when it is intended to give the work a superior appearance and also to conceal defects in inferior work.

GENERAL RULES FOR STONE SETTING

10. In assembling or putting together the stones to form a structure, certain principles or rules must be observed; namely:

1. That the vertical joints in any course should not come directly over the vertical joints in the course below.

2. That where the thickness of a wall is made up of two or more pieces of stone, bond stones or blocks that run through from face to face of the wall should be used whenever possible, for the purpose of binding the whole mass together.

3. That where the width of the wall is so great that a long bond stone would be liable to fracture, headers should be used at frequent intervals, placed as nearly over the center of the stretchers as possible, and extending two-thirds across the wall, alternately from opposite faces.

4. That when stratified stones are used, they should be laid on the natural bed, that is, the bed on which they rested in the quarry. Stratified stones when placed vertically are split and scaled by the action of the weather; moreover, a stone in this position has not as much strength to resist crushing as it has when placed with the lamina horizontal. Stones placed with their strata vertical can sustain only six-sevenths of the load borne by similar stones placed on the natural bed. When a stratified stone is used in a cornice with overhanging moldings, however, the natural bed should be placed parallel to the side joints; for, if placed horizontally, layers of the overhanging portions will be liable to drop off. In building arches, the stones should be set with the bed as nearly as possible at right angles to the thrust.

(Precise directions for ascertaining the natural bed of a stone cannot be given. With some stones it is easy to distinguish; with others, it is a matter of extreme difficulty; in case of doubt, the quarry owners should be consulted.)

5. That every joint or space between the stones should be filled with mortar, and that the spaces should be as small as possible.

6. That the surfaces of porous stones should be moistened with water, before being placed in contact with the mortar; otherwise they will absorb the moisture from the mortar, causing the mortar to become a friable mass.

7. That, for the sake of appearance, the largest stones should be placed in the lower courses, the thickness of the courses gradually decreasing toward the top.

8. That the rougher the beds and joints, the better the mortar should be. The principal office of the mortar is to equalize the pressure, and the more nearly the stones are dressed to closely fitting surfaces, the less important the quality of the mortar; with rough beds, the best quality of mortar should be used. This rule is frequently incorrectly reversed; that is, with fine, smooth, dressed beds, the best quality of mortar is used. When using stones that have been sawed, it may be necessary to roughen the surface of beds and joints with the point or tooth ax, so that the mortar will adhere.

9. That porous stones should not be placed at or below the ground line.

10. That, in foundations, absorption of moisture from below should be prevented by placing a course of impermeable material at or near the surface of the ground.

11. That porous stone should not be employed for copings, cornices, window sills, or other parts of a structure where water is likely to lodge.

12. That, in setting cut stones, as sills, water-tables, belts, etc., the mortar should be kept back about 1 inch from the face, the space being filled when the pointing is done.

13. That, if a stone that has once been set requires to be moved for any reason, it should be lifted clear from the

mortar bed, the mortar removed, and the stone set in a new bed of mortar in the new position.

14. That hammering or cutting stones on the top of stones just set in the work should not be practiced.

15. That all courses that project beyond the general lines of the wall, as sills, lintels, belt courses, etc., should be covered with boards or otherwise protected from damage.

RUBBLE MASONRY

11. Classification.—As stated in Art. **2**, **rubble masonry** is formed of irregularly shaped stones as they

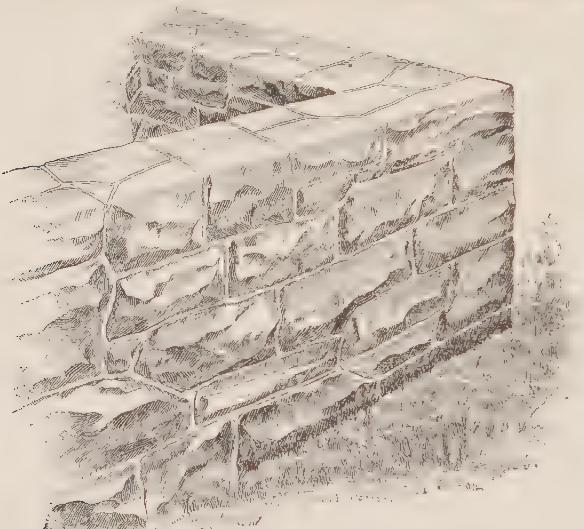


FIG. 6

come from the quarry, without other preparation than the removal of acute angles and excessive projections. Rubble masonry is either *uncoursed* or *coursed*. In **uncoursed masonry**, Fig. 6, the stones are laid without any attempt at regular courses. In **coursed masonry**, Fig. 7, the stone is leveled off at specified heights to an approximately horizontal surface. These courses are not necessarily of the same height throughout, but may rise by steps, in which case the work is termed **random coursed**.

12. Directions for Construction.—In building either class of rubble, the stone should be prepared by knocking off the thin edges. It should be cleansed from dust and dirt, and moistened with water, before it is placed in the structure. The stones should be placed on their widest beds, so that they may not be crushed or act as wedges and force out the adjacent work. The side joints should not form an angle with the bed joint sharper than 60°. The bonding of rubble masonry is secured by placing the largest

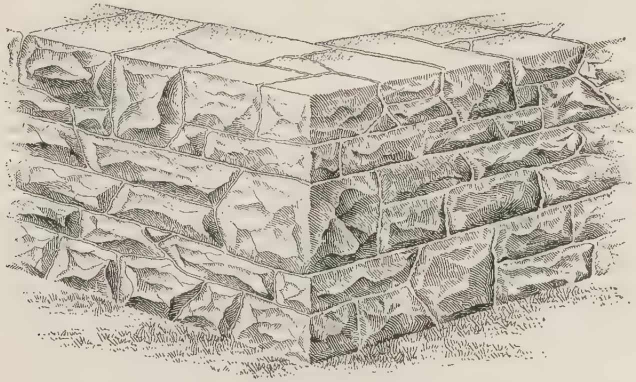


FIG. 7

stones at hand across the wall at frequent intervals. These large stones should form at least one-fourth of the face.

13. As the mortar governs to a great extent the strength of rubble masonry, it should be of good quality. The amount of mortar required varies according to the size of the stone. With small, irregular stones, one-third of the mass should be mortar. Rubble masonry is sometimes laid without mortar, as in retaining walls of small height, slope walls, paving, etc. When so laid, it is termed **dry rubble.**

14. Specifications for Rubble Masonry.—Rubble masonry should be formed of stones having a volume of not less than 1 cubic foot, and a thickness of not less than 6 inches. They must either have naturally fair beds or be roughly hammered thereto. The stones should be wet with water, then set on the natural bed in a full bed of mortar,

and hammered into place. The use of spalls in the bed joints should not be permitted. In the vertical joints, spalls may be allowed; the joint should be filled with mortar, and the spalls shoved in and hammered until solid. Bond stones should be placed so as to break joints, and should not be farther apart than 6 feet horizontally and 2 feet vertically.

SQUARED-STONE MASONRY

15. Classification.—Squared-stone masonry is

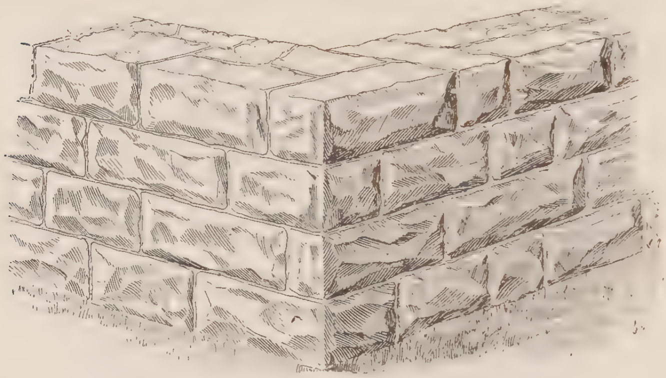


FIG. 8

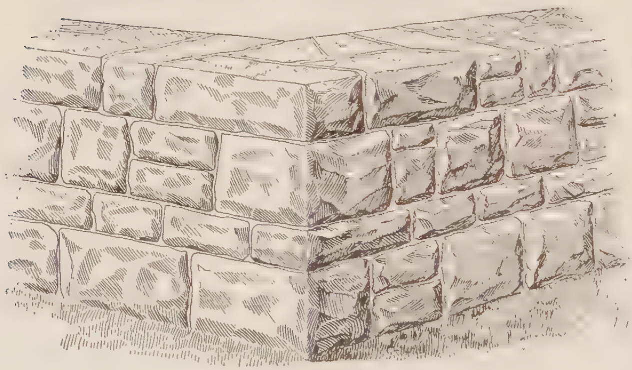


FIG. 9

formed by blocks of stone roughly hewn to a rectangular shape. It is divided into three classes; namely: **range**

work, Fig. 8, in which the courses are of the same height or rise throughout; **broken range,** Fig. 9, in which the heights of the courses are not uniform; and **random range,** Fig. 10, which is composed of rectangular blocks of different dimensions.

In all classes of squared-stone masonry, the stones are laid in thick beds of the best quality of mortar with side joints ranging from $\frac{1}{2}$ to 1 inch in width. The face of the

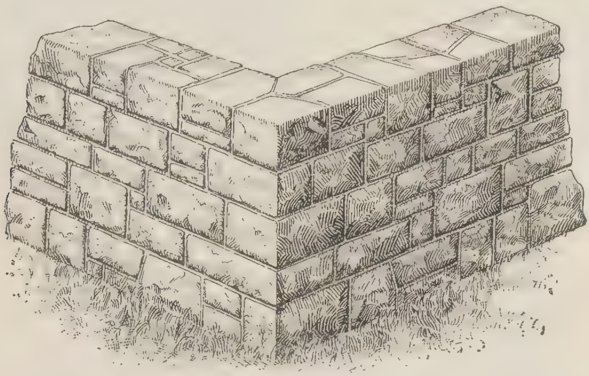


Fig. 10

stone is usually left as it comes from the quarry. In massive work, the edges are generally pitched to line.

16. Specifications for Squared-Stone Masonry. The stone should be dressed to a uniform thickness that, for any course, must not be less than 8 inches. The face should be quarry face, with edges pitched to a straight line. The joints should be broken by a lap of at least 8 inches, and there should be at least one header to every three stretchers. The stone should be laid in a full bed of mortar, and the width of the side joints should not exceed 1 inch.

ASHLAR MASONRY

17. Classification.—Ashlar masonry is divided into two classes; namely: **ranged** or **regular coursed ashlar,**

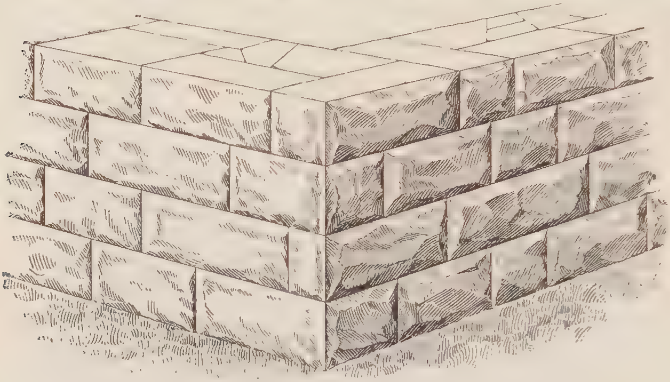


FIG. 11

Fig. 11, also called **cut-stone work** and **dimension-stone work,** which consists of rectangular blocks cut and dressed

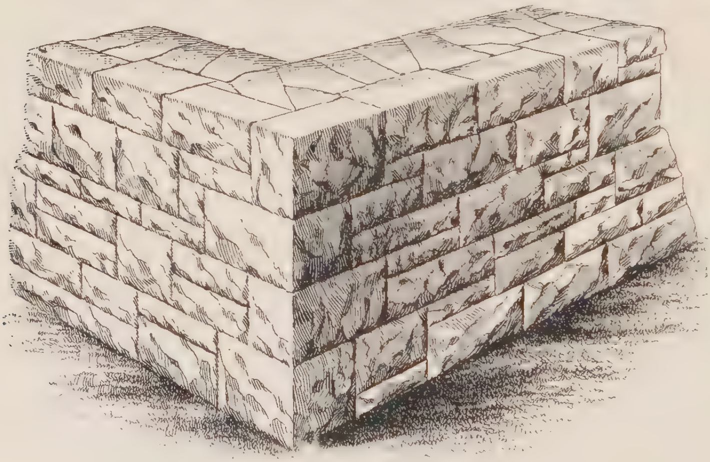


FIG. 12

to prescribed dimensions and built in courses of uniform height or rise; and **broken ashlar,** Fig. 12, which is formed of rectangular stones cut to dimensions, but in which the

stones are of unequal heights and are laid in the wall without any attempt at maintaining courses of equal rise.

18. Method of Construction.—Ashlar is considered the finest class of masonry, and is employed in all important structures. Its strength depends on the dimensions of the blocks, the accuracy of dressing, and the bond. The dimensions of the blocks are regulated by the character of the stone used. With sandstone and limestone, the length should not be greater than three times the rise, and the breadth should not exceed twice the rise; that is, if the rise or height of the stone is 1 foot, the length should be 3 feet, and the width or breadth 2 feet. With granite and the harder rocks, the length may be four times the rise, and the breadth,

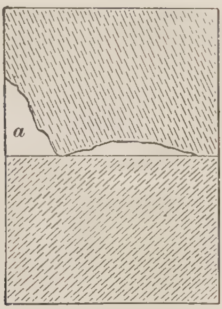


Fig. 13

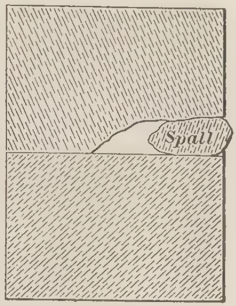


Fig. 14

three times the rise. The dressing consists in cutting the sides and bed joints to plane surfaces at right angles to one another; these surfaces should not be made smooth, but should possess a moderate degree of roughness, which aids the adhesion of the mortar and increases the resistance to displacement by sliding.

The accurate dressing of the beds to a plane surface is exceedingly important. If the surface is left *convex*, the convex portion will bear an undue amount of the pressure, and will form a joint called an **open joint**, which will gape at the edges. If the surface is made *concave*, the pressure is concentrated on the edges of the stone, causing them to *chip* or *spall* off, as shown at *a*, Fig. 13, thus causing a defacement

of the work, if not more serious damage. Concave beds are frequently made intentionally for the purpose of giving a neat appearance to the work; this practice must be carefully guarded against by close inspection. Another practice for securing close joints is to fairly dress the bed for a few inches from the face and then to chip away the stone toward the back, so that when the block is set it will be in contact with the adjacent stones only throughout this limited surface, as shown in Fig. 14. This practice is objectionable; it gives an inadequate extent of bearing surface to resist the pressure, thus causing the block to fracture. To give such a block a fair set, it has to be propped up with *spalls*, as shown in Fig. 14. This operation is called **spalling up, pinning up,** and **underpinning.** These props cause the pressure on the block to be thrown on a few points, and may cause the fracture of the stone.

19. The bed on which the stone is to be laid must be thoroughly cleaned of dust and be well moistened with water. A thin bed of mortar should then be spread evenly over it, and the stone, the lower edge of which has been cleaned and moistened, raised into position, and lowered first on one or two strips of wood, laid on the mortar bed; next, by the aid of the pinch bar, moved exactly into place, and truly plumbed; the strips of wood are then removed and the stone settled in place and leveled by striking it with a wooden mallet. In using bars and rollers for handling cut stone, the mason must be careful to protect the stone from injury by means of a piece of old bagging, carpet, etc.

20. The thickness of the mortar in well-executed ashlar should not exceed $\frac{1}{2}$ inch. In practice, it ranges from $\frac{1}{8}$ to $\frac{5}{8}$ inch.

21. The strength of ashlar depends, to a great extent, on the bond. The stones should overlap or break joint to an extent from one to one and one-half times the rise of the course; this, however, is rarely done, owing to the great cost. The common method is to lay up three or four courses of stretchers, with a header placed every 4 or 6 feet,

then to lay a course of headers. The total number of headers should be so proportioned that at least one-third of the face area shall be composed of headers.

22. The ashlar blocks usually form one face of a wall, or, in the case of a pier, they form the face of the two sides and ends, thus leaving a hollow space that must be filled up. This filling is called the **backing**. It is composed of either rubble masonry or concrete, and should be carried up course by course with the face work. The tails of the headers or bond stones should extend a considerable distance into the backing, in order to tie backing and face together.

23. **Broken Ashlar.**—Broken ashlar is cheaper than regular coursed ashlar, because, owing to the diversity in the size of the stones used, the entire output of the quarry can be worked up. When well executed, it presents a very pleasing appearance; it is extensively used for retaining walls and in situations where a massive appearance is not essential. In its construction, the same rules should be observed as stated for regular coursed ashlar.

24. **Specifications for Ashlar Masonry.**—The stone shall be granite of a fine and uniform grain, uniform in color, perfectly sound and free from sap, seams, cracks, or any other defect likely to impair its strength, durability, or appearance.

The face finish shall be rock face; it shall be uniform in appearance, and must not project more than 4 inches beyond the pitch lines; it must average at least $1\frac{1}{2}$ inches beyond the pitch lines, and have no depression extending back of them.

The upper and lower beds of each stone shall be rough-pointed, and cut true and parallel with each other, and must be true to the straightedge when applied in any direction. No hollow or slack cutting, nor falling away toward the back of the stone will be allowed.

The vertical joints must be full and square to the beds and to the face, for at least 12 inches back from the pitch lines, and beyond that must not open more than 2 inches.

The bottom bed shall always be the full size of the stone, and no stone shall have an overhanging top bed.

The bed joints shall be $\frac{1}{2}$ inch throughout, and the vertical joints of the face, for a distance of 12 inches back from the pitch lines, shall not exceed $\frac{1}{2}$ inch in thickness.

The masonry shall be laid in regular courses, and must be thoroughly bonded throughout. No stone in one course shall overlap the stones of the course next below by less than 15 inches. The courses shall be from 2 feet 6 inches to 2 feet in thickness. The thickest course shall be at the bottom, and the others shall decrease in thickness from the bottom toward the top.

Stretchers shall have a length of not less than twice nor more than four times their height, and a width of not less than 3 feet.

Every second or third stone in the face of each course shall be a header. Each header must be at least 3 feet wide on the face, and its length shall be at least three times its height.

No plug holes more than 9 inches in diameter or more than $1\frac{1}{2}$ inches deep, or within 1 foot of each other or from the edge of a stone, will be allowed.

25. All stones must be carefully cleaned and wet before being laid; stones already laid must be cleaned and moistened to receive the mortar bed for each stone to be laid on them. All stones shall be laid on their natural beds, in full flush beds of mortar, mixed fresh. All vertical joints must be thoroughly flushed with mortar. The face masonry must be kept clean at all times.

The stones must be lifted by a derrick and lowered into place, without shock to the stones already set. Any stone whose bond or set is disturbed by a neglect of this requirement must be taken up and reset at the cost of the contractor.

No grab holes shall be made in the finished faces of any coping or molded stone. Cutting of stones will not be permitted after they have been brought on the wall.

Water-tables, belt courses, pedestals, and coping shall be of granite, dressed with 6-cut work on the exposed faces;

the beds and end joints shall be fine-pointed. The width of the joints shall not exceed $\frac{1}{2}$ inch.

26. Face joints shall be cleaned out to a depth of $1\frac{1}{2}$ inches, and filled and pointed during mild weather, with mortar, which shall be driven in hard with a calking iron. The surface of the joints shall then be rubbed smooth with a rounded tool.

The mortar shall be composed of Portland cement and sand, in the proportion of 1 barrel of cement, weighing 375 pounds, net, to $8\frac{1}{2}$ cubic feet of sand. The cement and sand shall be thoroughly mixed dry. Clean, fresh water shall then be added, but only in quantities sufficient to give the proper consistency to the mortar where the mixing is complete.

BRICK MASONRY

DEFINITIONS AND GENERAL EXPLANATIONS

27. Bricks are usually placed in the wall with the largest side laid horizontally; hence, these sides are called the **beds**, and the smaller sides at right angles to them are

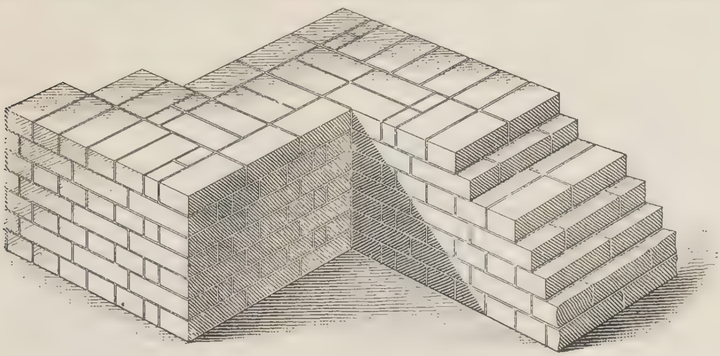


FIG. 15

called the **joints**. Occasionally, the bricks are laid with the smallest side horizontal, in which position the brick is called a **rowlock**. Courses composed of rowlocks are frequently used as the finishing course, and also to make up deficiences in height due to thin mortar beds Bricks whose long faces

lie parallel to the face of the wall are called **stretchers,** and bricks whose long faces lie at right angles to the face of the wall are called **headers.**

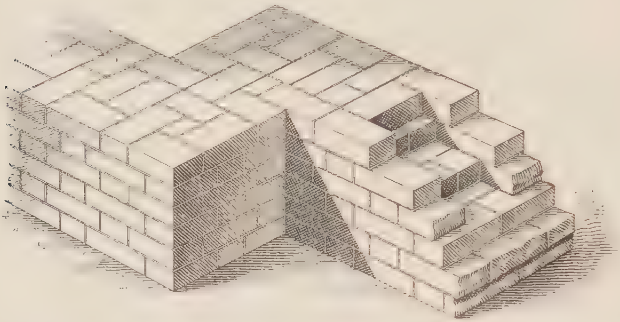


FIG. 16

28. A **course** is a horizontal layer of bricks. **Heading courses** are those showing no bricks but headers in the face of the wall.

29. **Bats** are pieces cut from bricks. A bat is called a one-quarter, one-half, or three-quarter bat, according to the proportion it bears to a whole brick.

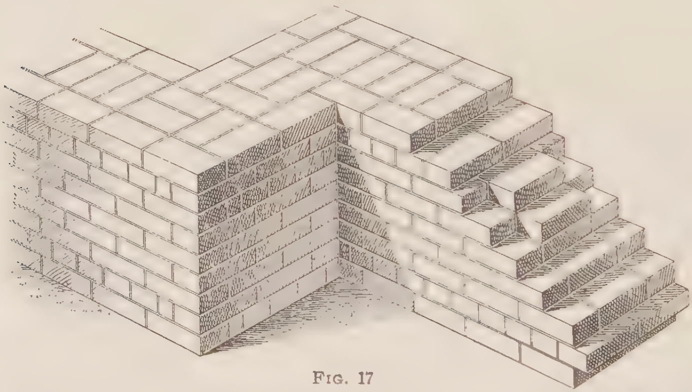


FIG. 17

30. **Bonding.**—Since bricks are made of uniform size, they are laid in the wall according to a uniform system called **bonding,** adapted to tying all parts of the wall together. There are several bonding systems, but only three are

generally used; namely, *English*, *Flemish*, and *common*. In all the systems of bonding, the aim is to prevent the vertical joints from forming a continuous straight line. To attain this object, the bricks are so laid that each vertical joint occurs directly over the center, or nearly so, of the brick in

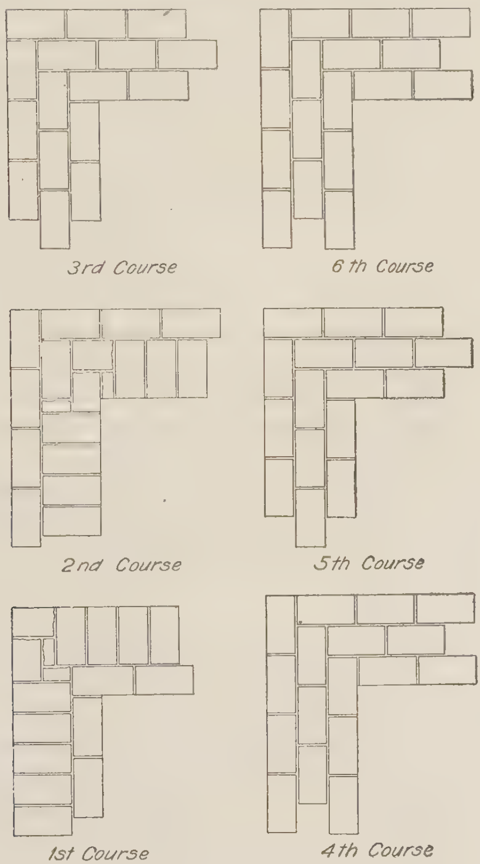


FIG. 18

the course below. This is called **breaking joints.** To secure a continuance of the proper relation between the vertical joints in each course, it is necessary to use, in several of the courses, pieces of brick, which, when used for this purpose, are called **closers.**

English bond, Fig. 15, consists of alternate courses of headers and stretchers. **Flemish bond,** Fig. 16, consists of alternate headers and stretchers in every course. **Common bond,** Fig. 17, consists in starting the wall with a course of headers, then laying up several courses (either three, five, or seven) of stretchers, followed by a course of headers. This arrangement is duplicated throughout the height of the wall.

The manner of arranging the courses to form the corner bond for a 13-inch wall is shown for six courses in Fig. 18. The same method is employed for walls of other thicknesses. The corners and angles should be built with care and by an experienced mason, because from them are stretched the lines to guide the laying of the courses between them. Acute and obtuse angles should be formed by bricks molded to the required angle. Intersecting walls are either bonded together, course by course, as shown in Fig. 19, or butted and anchored, as shown in Fig. 20 (*a*). A cross-section of the anchor is shown in Fig. 20 (*b*). It is composed of a flat bar bent at right angles at each end.

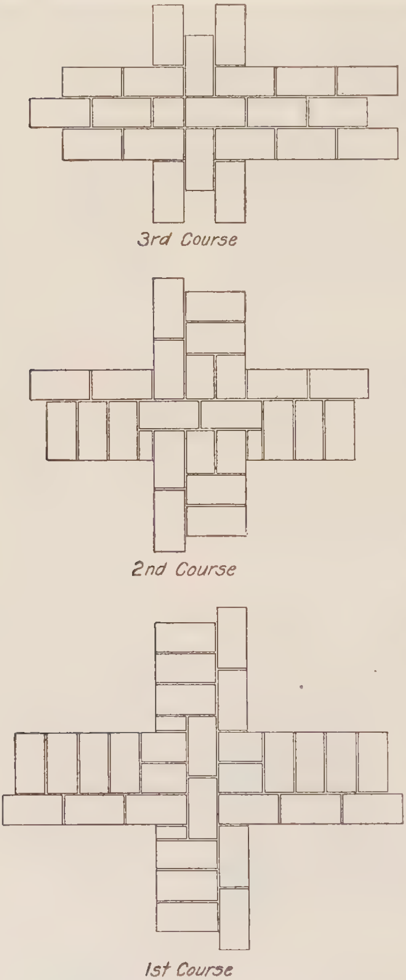


FIG. 19

31. Thickness of Walls.—The thickness of the wall is the distance from the face to the back, and is generally expressed in even inches, as 8 inches, or having a single brick; 12 inches, or a brick and a half; 16 inches, or two bricks; and so on. In reality, the 8-inch wall will measure $8\frac{1}{4}$ or 9 inches; the 12-inch, $12\frac{3}{4}$ or 13 inches; the 16-inch, 17 or more inches; the 20-inch, $21\frac{1}{4}$ or more inches; the difference being caused by the varying width of the bricks and the mortar joints between them.

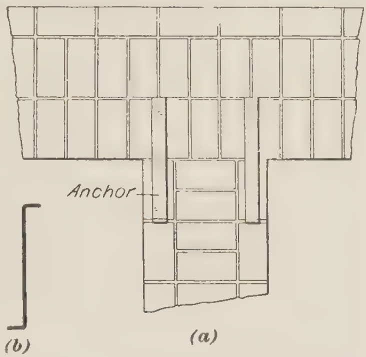


Fig. 20

BRICKLAYING

32. General Rules.—In the construction of brick masonry, the following general rules should be observed:

1. That the bricks should be sound and well shaped.

2. That the mortar should be of good quality, as the strength of brickwork depends largely on the quality of the mortar.

3. That each brick should be laid in a full bed of mortar, filling end and side joints at one operation. This operation is a simple and easy one for a skilful mason, but it requires constant watching to get it done. Masons have a habit of laying the brick in a bed of mortar, leaving the vertical joints to take care of themselves, throwing a little mortar over the top beds and giving a sweep with the trowel, which more or less disguises the open joint. Another thing that they often do is to draw the point of the trowel through the top bed of the brick, after the mortar has been applied, thus making an open channel with only a sharp ridge of mortar on each side, so that if the succeeding brick is taken up it will show a clear hollow, free from mortar, through the bed, as shown in Fig. 21. This enables them to bed the next brick with more facility,

and to avoid pressing on it to obtain the requisite thickness of joint.

4. That each brick should not be merely laid, but should be shoved and pressed into the mortar.

5. That each brick should be thoroughly cleaned and wetted before being laid. This is absolutely essential during hot weather. Bricks have a great avidity for water; if laid in the mortar dry, they will absorb the water, thus preventing the mortar from setting: the mortar will dry up and crumble in the fingers when handled. The common method of wetting bricks by throwing water from buckets or spraying with a hose over a large pile is deceptive: the water reaches only a few bricks on one or more sides. Immersion of the bricks for about 3 minutes is the only sure method of averting the evil consequences of using dry or partially wetted bricks.

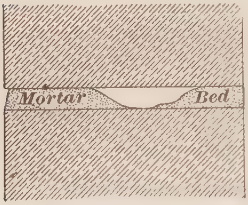


FIG. 21

6. That the first course should be started level. As bricks are of equal thickness throughout, the slightest irregularity or incorrectness in starting will be carried into the super-imposed courses. This can only be rectified by using at some places more or less mortar than should be used, and this is of course injurious to the work.

7. That the courses should be laid perpendicular, or as nearly so as possible, to the direction of the pressure they have to bear.

8. That the bricks in each course should break joints with those of the courses above and below by overlapping to the extent of from one-quarter to one-half the length of a brick.

9. That the two faces of the wall should be properly bonded or tied together. Every fifth or seventh course should be a header course. A common but improper method of building thick brick walls is to lay up the outer stretcher courses between the header courses, and then to throw mortar into the trough thus formed, making it semifluid by the addition of a large quantity of water, then throwing in the

bricks (bats, sand, and rubbish are often substituted for bricks), allowing them to find their own bearing; when the trough is filled, it is plastered over with stiff mortar and the header course laid, and the operation repeated. This practice may have the advantage of celerity in executing the work, but it weakens the strength, and is the cause of many failures.

33. Amount of Mortar Required.—The objects in using mortar are to cause the bricks to adhere and to furnish a uniform bearing, by filling up irregularities. The thickness of the mortar bed need not be more than will successfully accomplish these ends. In practice, it is usual to make exterior joints from $\frac{1}{4}$ to $\frac{3}{8}$ inch thick, and interior joints from $\frac{3}{8}$ to $\frac{1}{2}$ inch thick. The proportion of mortar to brick will vary with the size of the brick and the thickness of the joint. With the standard brick, $8\frac{1}{4}$ in. $\times$ 4 in. $\times$ $2\frac{1}{4}$ in., the amount of mortar required will be as given in the following table:

MORTAR REQUIRED IN BRICKWORK

Thickness of Joints Inch	Mortar Required Cubic Yard	
	Per Cubic Yard of Masonry	Per Thousand Bricks
$\frac{1}{2}$ to $\frac{5}{8}$	.3 to .40	.80 to .9
$\frac{1}{4}$ to $\frac{3}{8}$	.2 to .30	.40 to .6
$\frac{1}{8}$	.1 to .15	.15 to .2

34. Tools Used in Brick Masonry.—The principal tools used by the bricklayer are: the **trowel**, Fig. 22, used for lifting and spreading the mortar; the **hammers**, shown in Figs. 23, 24, and 25, used for cutting hard bricks (soft bricks are usually cut with the edge of the trowel); the **jointers**, Fig. 26 (*a*) and (*b*), used for pointing the mortar joints; the **plumb-rule**,

FIG. 22

Fig. 27, used to test the verticality of walls as they are carried up; and the **cold chisel,** Fig. 28, used for cutting through walls, making chases, etc.

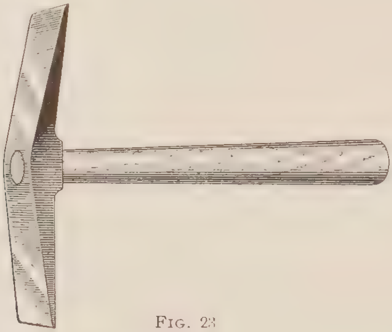


FIG. 23

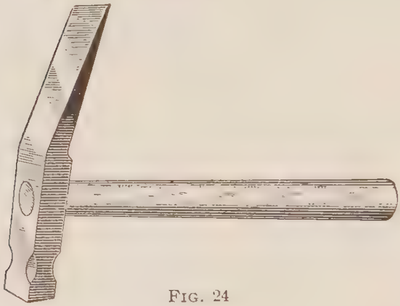


FIG. 24

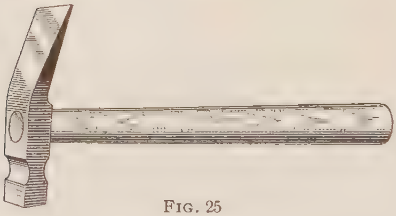
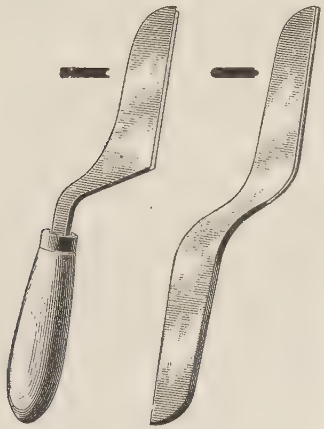


FIG. 25

SPECIFICATIONS FOR BRICK MASONRY

35. The bricks shall be of the best quality of hard burnt bricks, burnt hard entirely through, and regular and uniform in shape and size. They shall be of compact texture. Bricks that, after being thoroughly dried and immersed in water for 24 hours, absorb more than 16 per cent. in volume

of water may be rejected. Bricks to be used in the face work shall have even, straightedged faces. The brick masonry shall be laid, as required, in natural- or in Portland-cement mortar, and the cément shall be mixed with one part or two parts of sand, as required. The bricks are to be thoroughly wet just before laying. Each brick shall be completely embedded in mortar under its bottom, on its sides,



(a) (b)

FIG. 26

FIG. 28

FIG. 27

and on its ends at one operation, care being taken to fill every joint. No bats shall be used except where whole bricks cannot be used, and nothing smaller than a half brick shall be used in making closures. All work must be well and thoroughly bonded. The joints shall have such thicknesses, not less than $\frac{1}{4}$ inch, as the engineer shall direct. The face joints shall be neatly struck or pointed when the bricks are laid.

PRESERVATION AND REPAIRING

36. Brick Masonry Impervious to Water.—As brick masonry is naturally very permeable, it is desirable under some conditions to render it waterproof. Many processes for this purpose have been invented and experimented with, but few of them have been found effective. Laying the bricks in asphaltic mortar, coating the walls with asphalt or coal tar, and the application of "Sylvester's wash" have proved fairly successful. The latter process consists in using two washes for covering the surface of the walls, one composed of Castile soap and water, and one of alum and water. These solutions are applied alternately until the walls are made impervious to water.

37. Efflorescence is the name given to a grayish-white substance that appears on the surface of brick walls, particularly in moist climates. It arises from several causes, and often returns after being removed. It is generally considered to originate in the mortar, being the precipitate resulting from the solution by rain water of the salts of soda, potash, magnesia, etc. contained in the lime or cement. In some cases, it has been found to proceed from the bricks when they have been burned with sulphurous coal or made from clay containing iron pyrites.

Many remedies have been tried for efflorescence, such as: (*a*) adding baryta to the water used for tempering the brick clay, or to the mortar; (*b*) washing the surface with diluted muriatic acid, and then applying a coating of linseed oil; (*c*) using Sylvester's washes, composed of the same ingredients and applied in the same manner as for rendering masonry impervious to moisture.

38. Repair of Masonry.—In effecting repairs in masonry, when new work is to be connected with old, the mortar of the old should be thoroughly cleaned off along the surface where the junction is to be made, and the surface thoroughly wetted. The bond and other arrangements will depend on the circumstances of the case. The

surfaces connected should be fitted as accurately as practicable, so that, by using but little mortar, no disunion may take place from settling. As a rule, it is better that new work should butt against the old, either with a straight joint visible on the face, or let into a chase, sometimes called a **slip joint,** so that the straight joint may not show; but, if it is necessary to bond them together, the new work should be built in a quick-setting cement mortar, and each part of it allowed to set before being loaded.

In pointing old masonry, all the decayed mortar should be completely raked out with a hooked iron point, and the surfaces should be well wetted before the fresh mortar is applied.

PLAIN CONCRETE

INTRODUCTION

1. Concrete consists of cement in a plastic state mixed with small fragments of some hard material, called an **aggregate.** The plastic cement, either by itself or with the sand that is used in all concrete, is called the **matrix.** The concrete used today is almost exclusively made with hydraulic cement, wet with water to make it set and adhere to the aggregate. For this reason, the term *concrete*, as commonly used, refers only to that variety. When it is desired to specify any other kind of concrete, it is usual to mention it by its full name, as *bituminous concrete* or *lime concrete*. Such varieties, however, are of comparatively little importance, and will not be treated here.

The term *concrete*, besides being restricted to hydraulic-cement concrete, has another restriction: the aggregate must not be sand alone, although it may be partly sand. A mixture of hydraulic cement, sand, and water is called by the special name of **mortar.**

2. Materials Used for Aggregates.—The aggregate is used simply to cheapen concrete. Pure, or **neat,** cement, when wet with water, would fulfil all the physical requirements of concrete, but, on account of the price of cement, would be too expensive. The aggregate used today is generally a mixture of sand and broken stone. To it are often added small chips, and even fine grains coming from the stone crusher. Sometimes, gravel and sand are used as an aggregate; but, on account of the smooth surface of the

gravel pebbles, the cement does not adhere to them as strongly as it does to the rough faces of broken stone. Cinders are used instead of broken stone when it is desired to make a concrete of lighter weight. Broken bricks are also sometimes used. Sand and sea shells are used for concrete in localities where shells abound, and make an article of superior quality. The three principal concretes used, however, are those made of cement, sand, and broken stone; cement, sand, and gravel; and cement, sand, and cinders.

3. Theory of Mixture.—As already stated, the aggregate is used to cheapen the concrete. The function of the cement is to form a solid mass with the aggregate. The strength of concrete depends very largely on the strength of the cement used in its manufacture, and so it is useless to try to make good concrete with poor cement. The best concrete is that in which the voids or spaces between the grains of sand are completely filled with cement, and in which the mixture of cement and sand completely fills the voids in the broken stone.

4. Uses of Concrete.—The value of concrete as a substitute for stone and brick has been known for many centuries, and is to be seen in the ruins of ancient Roman temples and palaces, in domes and arches, in the interior of brick-faced walls, and in their foundations. The great advantage of concrete over natural stone is that it does not have to be cut to size. A large amount of labor is expended in cutting stones to fit in their required place in masonry construction. This expense is saved by using concrete. Concrete is, in fact, a plastic stone that will assume any form or fill any mold, and will become as hard as natural stone when the cement in it has set.

5. A kind of masonry that is often used today is known as **concrete rubble.** It consists of concrete into which large, sound, uncut stones are placed, usually with the aid of a steam derrick. It holds a place midway between ordinary concrete work and rubble masonry. The object of its use is to cheapen the entire structure by saving concrete. If the concrete is wet enough to come into perfect contact with the

large stones, this kind of masonry will be found equal to pure-concrete construction.

6. The uses of concrete are innumerable. The building of foundations at present forms one of its principal fields, but arches, sewers, culverts, dams, roads, and many other works are constructed of this material. Since the introduction of reinforced concrete, the field has broadened immensely: water mains, railroad ties, beams, columns, piles, bridges, and roofs may be added to the list.

7. General Statement of the Method of Making Concrete.—For all the uses of concrete, the methods of manufacture are almost the same in principle. The wet cement, sand, and broken stone are thoroughly mixed and put into a mold, usually made of wood. This mold is removed after the cement has set, when the concrete is ready for use. The details of this process will be explained in subsequent articles.

MATERIALS AND THEIR PROPORTIONS

QUALITY OF MATERIALS

8. Cement.—The cement used for concrete work is generally Portland cement. Natural cement is used less and less in proportion every year. This latter cement is not as strong and reliable as Portland. It sets more quickly, but takes longer to obtain its ultimate strength. It is used where economy demands it, but should never be placed under water. In civil-engineering work it is seldom employed, except in the form of mortar. A very good substitute for Portland cement in concrete for use under water is *pozzuolana cement.* This cement never gets very hard, but it withstands the action of sea-water even better than Portland cement. It will, however, soon fail if subjected to much attrition and wear.

9. Sand.—The subject of sand has already been treated in *Cementing Materials and Mortar.* The sand used for concrete

should be sharp and free from loam and chemical salts, particularly salts of a hygroscopic nature. The sand should not be too fine. An investigation made by A. S. Cooper on the effect that the size of the grains of sand has on the strength of mortar led him to the conclusion that, up to a certain limit, mortars become stronger as the grains of sand used become larger. However, the amount of cement required to fill the voids between the grains of sand is an item of importance, and increases with the size of the grains themselves. It is, therefore, customary to use sand with some coarse grains in it, but with enough smaller grains to fill the voids between the larger ones.

10. Broken Stone and Other Aggregates.—The proper size of stone to be used for an aggregate will be considered later. There are two other important points to be mentioned here concerning aggregates; they refer to the fire-resisting and to the steel-protecting qualities of the concrete. When concrete is to be used in a place where it may have to withstand the action of fire, it is necessary that the aggregate be of such nature that it will not disintegrate and crumble away. Limestone and marble chips are objectionable as aggregates, as the action of heat causes them to swell, crack, and crumble to dust. Trap rock, cinder, and broken brick are among the best aggregates for concrete that is to be exposed to the action of fire. It should be borne in mind, however, that broken brick will soon soften in concrete placed under water.

11. Limestone is an uncertain substance to place against steel in reinforced concrete work. Some years ago, the strands of the main cables embedded in the concrete in the anchorages of the Niagara railroad bridge were found in some places to be eaten almost entirely through. Some engineers are of the opinion that this corrosion was caused by the limestone aggregate used. It is, therefore, according to the best information obtainable, unsafe to use limestone in reinforced-concrete work, unless special care is taken to see that the steel is well protected from the stone by a layer of

cement. Another material that is considered injurious to steel, if the latter is not coated with cement, is cinders; their damaging effect is not due so much to the sulphur in them, as commonly claimed, as to their porosity. However, in certain proportions in which the cinder is not so predominant—as in a mixture of one part of cement, two parts of sand, and three parts of cinders—the corrosive effect on the steel is inconsiderable if the concrete is properly mixed.

PROPORTIONING OF INGREDIENTS

12. Effect of Proportion of Ingredients on Strength and Imperviousness.—There are two principal requirements that concrete should fulfil: it should be capable of withstanding pressures that cause direct compression in its mass, and it should be waterproof. In almost all concrete construction with which the civil engineer has to deal, one or the other or perhaps even both of these requirements are of paramount importance. Concrete is seldom used to withstand tension, because its tensile strength is very low.

The compressive strength of concrete depends on the strength of the cement and the thoroughness with which the cement binds together the various pieces of the aggregate. The more completely the voids are filled, the more completely will the aggregate be held together. Therefore, it may be stated that the more solid and condensed the concrete is, the less voids it will have, and the stronger it will be. The same is true with regard to making concrete waterproof: the more dense the concrete is, the more nearly waterproof it is. When it is desired to make the concrete more impervious to water, it is made richer in cement, in order that the voids in the sand and broken stone may be more completely filled. A mixture of 1 part of cement, $1\frac{1}{2}$ parts of sand, and 3 parts of stone, which would be considered extravagantly rich for a dry place, is probably as dense a concrete, and as good for waterproofing qualities, as can be made. It is, however, impossible to make concrete entirely

waterproof without the addition of some special coating, as will be explained further on.

13. Aggregates.—As already stated, the proper proportion of ingredients for the best concrete is such that there will be enough cement in the mixture to bind all the materials together, and that the materials will be of such various sizes that all voids will be filled. When a concrete is made of cement, sand, and stone, and the stone is of such a size that it will pass through a 3-inch ring, but will not pass through a $2\frac{1}{2}$-inch ring, the concrete is weaker and requires more cement than one made with graded stone from 3 inches down. When the stone is graded in size, the smaller-sized stones fill the voids between the larger stones, and thus reduce the amount of cement required. The grading of the stone also makes the concrete stronger. The cause of this increase of strength is not apparent, but numerous experiments have proved the fact without question.

It would seem that to make a perfect concrete there ought to be certain fixed proportions of ingredients, and fixed graduations in the size of the aggregate, starting with a certain largest size of stone, and then a certain amount of stone, say $\frac{1}{4}$ inch smaller, and so on down to the different grades of sand. This fact is undoubtedly true, but to mix the ingredients in the proper proportions is a difficult task, which should not be undertaken except in work of great magnitude and importance. In practice, the small stone chips (except the very fine stone dust, which is always considered injurious to concrete, and is usually screened out) resulting from the crushing of broken stone are usually added to the concrete. Some engineers advise that the chips be first screened out from the broken stones, and afterwards added in known proportions. This is another step in the right direction. If all these precautions were followed, it might be possible to make a concrete that would be absolutely reliable under given conditions, and with a crushing strength that could be foretold to within about 5 per cent.; but, on account of other conditions that enter

into the problem, such accuracy cannot be attained. It is, therefore, useless as a rule to try to do more than provide graded stone and graded sand for the concrete, without troubling so much about the proportions of the various sizes. The engineer decides the proportions of cement to sand and broken stone, and the maximum size of the latter, and that is as far as he usually goes.

14. It is customary, when specifying broken stone, to give a maximum allowable size. Some engineers specify that the stone must pass through a ring 2 inches in diameter; more engineers specify a $2\frac{1}{2}$-inch ring, and even a 3-inch ring is not uncommon. For very thin walls, and for small work, such as concrete blocks, it is necessary, of course, that the size of broken stones shall not be too large to place them in the mold. It can, however, be stated as a general proposition that the larger the stones, other things being equal, the stronger will be the concrete. This fact is clearly shown by Table I, which gives the results of tests made at the Watertown Arsenal in 1898, and published by the United States government. The general increase in strength with the increase in size of broken stone or gravel used will be noted. It is also interesting to know that the concrete becomes heavier per cubic foot, or, in other words, more dense, the larger the stone used. All these tests were made with concrete manufactured in the proportion of one part of cement, one part of sand, and three parts of broken stones, or a 1 : 1 : 3 (1 to 1 to 3) mixture, as it is usually expressed. It should be observed that the proportion of ingredients is customarily indicated in the form of a continued ratio consisting of three terms, the first of which indicates the amount of cement; the second, the corresponding amount of sand; and the third, the corresponding amount of broken stone or whatever takes its place. The figures on cinder concrete given in Table I are added simply to give a comparison of weights.

15. Proportioning by Voids.—In one method of proportioning concrete, the ratio of cement to sand is first

TABLE I

COMPRESSIVE STRENGTH OF CONCRETE MADE OF DIFFERENT-SIZED STONES

Material	First Group			Second Group			Third Group			Fourth Group		
	Age Days	Weight per Cubic Foot Pounds	Compressive Strength Pounds per Square Inch	Age Days	Weight per Cubic Foot Pounds	Compressive Strength Pounds per Square Inch	Age Days	Weight per Cubic Foot Pounds	Compressive Strength Pounds per Square Inch	Age Days	Weight per Cubic Foot Pounds	Compressive Strength Pounds per Square Inch
Trap $\frac{1}{2}$ inch . .	7	145.56	1,391	19	149.00	2,220	32	146.44	2,800	76	153.34	5,021
Trap $\frac{3}{4}$ inch . .	8	147.01	1,900	20	150.12	2,769	32	148.27	3,200			
Trap 1 inch . .	7	159.26	3,390	20	160.65	4,254	34	160.88	4,917	73	158.54	5,272
Trap $1\frac{1}{2}$ inches .	11	157.80	3,189	26	160.56	4,006	41 48	161.14 157.39	4,562 2,583			
Trap $2\frac{1}{2}$ inches .	7	158.37	2,400	22	159.27	4,143	32	161.44	4,140	65	161.76	4,523
Pebbles $\frac{3}{8}$ inch .	7	146.76	1,298	21	150.51	1,298	34	147.02	2,992	70	148.76	3,870
Pebbles $1\frac{1}{2}$ inches	7	149.63	2,276	22	151.75	2,276	29	151.51	3,817	61	150.89	4,018
Pebbles 3 inches	11	151.36	2,800	26	150.63	2,800	41 46	153.60 153.21	4,200 3,400			
Cinders	31	115.90	2,329	90	114.12	2,834						

decided on; for example, a 1 : 3 mixture (one part of cement to three of sand). A certain volume of broken stone of a quality similar to that which it is proposed to use in the concrete is put in any vessel, as a barrel, of known capacity. The broken stone must just fill the vessel. Water is then poured on until the vessel holds no more, and this water, which has filled all the voids in the broken stones, is then run off and measured. The ratio of the volume of water to the entire volume of the vessel gives the proportion of the voids. Now, the mixture of the sand and cement, which is assumed to have the same volume as the sand alone, must fill the voids in the stone. Suppose, for example, that 40 per cent. of voids are found; it is customary to add 10 per cent. for inaccuracy, so that, in this case, the voids are 50 per cent. of the whole volume; therefore, 50 per cent. of sand must be added to fill the voids, and, consequently, the ratio of the volume of sand to that of stone is 1 : 2, or 3 : 6. Therefore, the proper proportion of the materials for the concrete under consideration would be 1 : 3 : 6 (one part cement, three parts sand, six parts stone). This method of proportioning concrete is not very accurate, and a better method is that previously described of proportioning the materials by size. As already stated, however, the latter method is comparatively complicated; it is used only for works of great magnitude and importance, and is usually left to a concrete specialist.

16. Another Method of Proportioning.—Still another method of proportioning the materials, and a method that is simple and fairly accurate, is as follows: A batch of concrete is mixed in known proportions. The same quantity of water is used that it is proposed to use on the work, and the mixture is rammed and *tamped* in the receptacle in the same manner as it is proposed to do on the work. The receptacle should preferably be of metal; a tin washtub, or a short section of 12-inch pipe, capped at one end, will answer. When the receptacle is full, it is weighed, and if the weight of the receptacle itself has previously been

found, the weight of the concrete may be obtained. Various other mixtures of concrete are tried in the same manner, and since the denser the mixture the stronger it will be, the heaviest concrete is the strongest for the particular work. It is to be remembered that each batch of concrete must be weighed and taken out of the receptacle before it has time to set, as otherwise some difficulty might be experienced in getting it loose.

17. Usual Proportions.—It is not always necessary to use the strongest concrete, as the concrete may be required to withstand only slight stresses and be simply used for its weight. The strongest concrete would then be unnecessarily expensive. It is seldom, therefore, that any of the above methods for proportioning concrete are employed, and the engineer usually specifies a mixture from his own experience without testing the aggregates in any way, except to see that the stone is under the specified maximum size and that the sand is in large grains and free from dirt and loam. A common proportion for unimportant work is 1 : 3 : 6. This proportion may be used for foundations below ground in engine bases, in the foundations for asphalt pavements, and for other similar purposes. A richer mixture, 1 : 2 : 4, is used in piers, in dams, in important reinforced-concrete work, and in other places where great strength is desired. The average ultimate strengths of such mixtures are given in Table III, which will be again referred to in a subsequent article.

18. Water.—The wetter the concrete is, the easier it will be to put in place, but mixtures that are too wet are not so strong as medium mixtures. The amount of water that will make the best mixture is such that after the concrete has been put in place and rammed it will quake like jelly when struck with a spade, and water will come to the surface. If the concrete is wetter than this, the water will have a slight chemical effect on the cement, and, moreover, the ingredients of the concrete will tend to separate, the broken stones settling to the bottom of the mold, and the cement rising to the top.

In cinder concrete, on account of the porosity of the cinders, it is necessary to use a little more water, so that the cement will be liquid enough to fill the little cavities in each cinder. This precaution is indispensable when the concrete is to be used with steel, as otherwise the steel will be rapidly corroded by the action of air reaching it through the pores in the cinder.

19. Dry Concrete.—With the advent of the concrete block, one hears a great deal about **dry concrete.** This name is given to concrete in which as little water as possible is mixed. In the concrete-block manufacturing business, the mold in which each block is made is required as soon as possible, so that it can be used over again, and thus increase the capacity of the machine to which it belongs. For this reason, the concrete-block manufacturers use as a rule dry concrete, and attempt to supply the remainder of the water required for the complete crystallization or setting of the cement by **curing** the blocks; that is, by sprinkling them with water for a week or so. Dry concrete, they claim, is stronger than concrete made with its full proportion of water. The results, however, of many recent tests made by well-known experimenters seem to indicate that dry concrete will show higher compression values for a limited time after it is made, but that the rate of increase of strength is not so great as with wet concrete. After 1 year or 6 months, the strength of the wet concrete will be found to have attained and perhaps surpassed that of the dry mixture.

There is a serious objection to dry concrete; namely, that it cannot be rammed to so dense a mass as wet concrete. Therefore, when it is desired to make concrete waterproof, it should be mixed wet. Too much water, however, must be avoided, for this will tend to make the cement float to the top of the mass.

MEASURING AND ESTIMATING INGREDIENTS

20. Methods of Measuring Ingredients.—After deciding what proportions of ingredients will be used for the concrete, the engineer must be able to calculate the exact quantity of each material that he must order. Cement is bought by the barrel, but is usually shipped by the bag. Four bags of Portland cement make a barrel. Natural cement comes in the same-sized bags, or in larger bags making 3 bags to a barrel. An ordinary box car holds from 400 to 600 bags. The purchaser is charged with the bags by the manufacturer, unless they are of paper, but he gets a rebate for those he returns. A barrel of Portland cement weighs 375 pounds; a barrel of natural cement, 300 pounds.

Cement is usually measured by the barrel the way it comes from the manufacturer, or as four bags to the barrel, while broken stone and sand are measured loose in a barrel. Portland cement, after it is taken out of its original packing and stirred up, fills a larger volume than when packed. It is, therefore, necessary to state just how the cement is to be measured; and, as said before, it is customary to measure it by the barrel compact. A cement barrel contains about 3.8 cubic feet.

21. Fuller's Rule.—A practical rule has been devised by W. B. Fuller whereby, after the proportions of ingredients have been fixed, the quantity of material for a certain work may be obtained. It is called **Fuller's rule for quantities,** and may be expressed in mathematical symbols as follows:

Let c = number of parts of cement;
s = number of parts of sand;
g = number of parts of gravel or broken stone;
C = number of barrels of Portland cement required for 1 cubic yard of concrete;
S = number of cubic yards of sand required for 1 cubic yard of concrete;
G = number of cubic yards of stone or gravel required for 1 cubic yard of concrete.

Then,
$$C = \frac{11}{c + s + g} \qquad (1)$$
$$S = \frac{3.8}{27} Cs \qquad (2)$$
$$G = \frac{3.8}{27} Cg \qquad (3)$$

If the broken stone is of uniformly large size with no smaller stone in it, the voids will be greater than if the stone were graded. Therefore, 5 per cent. must be added to each value found by the preceding formulas.

EXAMPLE.—If a 1 : 2 : 4 mixture be considered, what will be: (*a*) the number of barrels of cement, (*b*) the number of cubic yards of sand, and (*c*) the number of cubic yards of stone required for 1 cubic yard of concrete?

SOLUTION.—(*a*) Here, $c = 1$, $s = 2$, and $g = 4$. Substituting these values in formula **1**,
$$C = \frac{11}{1 + 2 + 4} = 1.57. \quad \text{Ans.}$$

(*b*) Substituting the values of C and s in formula **2**,
$$S = \frac{3.8}{27} \times 1.57 \times 2 = .44. \quad \text{Ans.}$$

(*c*) Substituting the values of C and g in formula **3**,
$$G = \frac{3.8}{27} \times 1.57 \times 4 = .88. \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. How much (*a*) Portland cement (*b*) sand (*c*) broken stone is required to make 1 cubic yard of 1 : 3 : 6 concrete?

Ans. { (*a*) 1.10 bbl. (*b*) .46 cu. yd. (*c*) .93 cu. yd.

2. A certain concrete foundation is to be 15 feet long, 15 feet wide, and 12 feet deep. If a 1 : $2\frac{1}{2}$: 5 mixture is used, how much: (*a*) cement will be required? (*b*) sand? (*c*) broken stone?

Ans. { (*a*) 129 bbl. (*b*) 45 cu. yd. (*c*) 91 cu. yd.

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22. Table of Quantities.—Table II, giving the quantities of ingredients for concrete of various proportions, has been prepared by Edwin Thacher. It will be noted that Mr. Thacher takes into account the difference in the character and size of the stone or gravel used.

TABLE II

QUANTITIES OF INGREDIENTS FOR CONCRETE OF VARIOUS PROPORTIONS

Proportion of Ingredients			Required for 1 Cubic Yard Rammed Concrete											
			Stone 1 Inch and Under—Dust Screened Out			Stone 2½ Inches and Under—Dust Screened Out			Stone 2½ Inches—With Most Small Stone Screened Out			Gravel ¾ Inch and Under		
Cement	Sand	Stone	Cement Barrels	Sand Cubic Yard	Stone Cubic Yard	Cement Barrels	Sand Cubic Yard	Stone Cubic Yard	Cement Barrels	Sand Cubic Yard	Stone Cubic Yard	Cement Barrels	Sand Cubic Yard	Gravel Cubic Yard
1	1.0	2.0	2.57	.39	.78	2.63	.40	.80	2.72	.41	.83	2.30	.35	.74
1	1.0	2.5	2.29	.35	.88	2.34	.36	.89	2.41	.37	.92	2.10	.32	.80
1	1.0	3.0	2.06	.31	.94	2.10	.32	.96	2.16	.33	.98	1.89	.29	.86
1	1.0	3.5	1.84	.28	.98	1.88	.29	1.00	1.88	.29	1.05	1.71	.26	.91
1	1.5	2.5	2.05	.47	.78	2.09	.48	.80	2.16	.49	.82	1.83	.42	.73
1	1.5	3.0	1.85	.42	.84	1.90	.43	.87	1.96	.45	.89	1.71	39	.78
1	1.5	3.5	1.72	.39	.91	1.74	.40	.93	1.79	.41	.96	1.57	.36	.83
1	1.5	4.0	1.57	.36	.96	1.61	.37	.98	1.64	.38	1.00	1.46	.33	.88
1	1.5	4.5	1.43	.33	.98	1.46	.33	1.00	1.51	.35	1.06	1.34	.31	.91
1	2.0	3.0	1.70	.52	.77	1.73	.53	.79	1.78	.54	.81	1.54	.47	.73
1	2.0	3.5	1.57	.48	.83	1.61	.49	.85	1.66	.50	.88	1.44	.44	.77
1	2.0	4.0	1.46	.44	.89	1.48	.45	.90	1.53	.47	.93	1.34	.41	.81
1	2.0	4.5	1.36	.42	.93	1.38	.42	.95	1.43	.43	.98	1.26	.38	.86
1	2.0	5.0	1.27	.39	.97	1.29	.39	.98	1.33	.39	1.03	1.17	.36	.89
1	2.5	3.5	1.45	.55	.77	1.48	.56	.79	1.51	.58	.81	1.32	.50	.70

1	2.5	4.0	1.35	.52	.82	1.38	.53	.84	1.42	.54	.87	1.24	.47	.75
1	2.5	4.5	1.27	.48	.87	1.29	.49	.88	1.33	.51	.91	1.16	.44	.80
1	2.5	5.0	1.19	.46	.91	1.21	.46	.92	1.26	.48	.96	1.10	.42	.83
1	2.5	5.5	1.13	.43	.94	1.15	.44	.96	1.18	.44	.99	1.03	.39	.86
1	2.5	6.0	1.07	.41	.97	1.07	.41	.98	1.10	.41	1.03	.98	.37	.89
1	3.0	4.0	1.26	.58	.77	1.28	.58	.78	1.32	.60	.80	1.15	.52	.72
1	3.0	4.5	1.18	.54	.81	1.20	.55	.82	1.24	.57	.85	1.09	.50	.75
1	3.0	5.0	1.11	.51	.85	1.14	.52	.87	1.17	.54	.89	1.03	.47	.78
1	3.0	5.5	1.06	.48	.89	1.07	.49	.90	1.11	.51	.93	.97	.44	.81
1	3.0	6.0	1.01	.46	.92	1.02	.47	.93	1.06	.48	.97	.92	.42	.84
1	3.0	6.5	.96	.44	.95	.98	.44	.96	1.00	.45	1.01	.88	.40	.87
1	3.0	7.0	.91	.42	.97	.92	.42	.98	.94	.42	1.05	.84	.38	.89
1	3.5	5.0	1.05	.56	.80	1.07	.57	.82	1.11	.59	.85	.96	.50	.76
1	3.5	5.5	1.00	.53	.84	1.02	.54	.85	1.06	.56	.89	.92	.48	.78
1	3.5	6.0	.95	.50	.87	.97	.51	.89	1.00	.53	.92	.88	.46	.80
1	3.5	6.5	.92	.49	.91	.93	.49	.92	.96	.51	.95	.83	.44	.82
1	3.5	7.0	.87	.47	.93	.89	.47	.95	.91	.49	.98	.80	.43	.85
1	3.5	7.5	.84	.45	.96	.86	.45	.98	.86	.47	1.01	.76	.41	.87
1	3.5	8.0	.80	.42	.97	.82	.43	1.01	.81	.45	1.04	.73	.39	.89
1	4.0	6.0	.90	.55	.82	.92	.56	.84	.95	.58	.87	.83	.51	.77
1	4.0	6.5	.87	.53	.85	.88	.53	.87	.91	.55	.90	.80	.49	.79
1	4.0	7.0	.83	.51	.89	.84	.51	.90	.87	.53	.93	.77	.47	.81
1	4.0	7.5	.80	.49	.91	.81	.50	.93	.84	.51	.96	.73	.44	.83
1	4.0	8.0	.77	.47	.93	.78	.48	.95	.81	.49	.98	.71	.43	.86
1	4.0	8.5	.74	.45	.95	.76	.46	.98	.78	.47	1.01	.68	.42	.88
1	4.0	9.0	.71	.43	.97	.73	.44	1.01	.75	.45	1.04	.65	.40	.89
1	5.0	9.0	.66	.50	.90	.67	.52	.93	.70	.53	.96	.61	.46	.83
1	5.0	10.0	.62	.47	.95	.63	.48	.96	.65	.50	1.00	.57	.43	.87

PROPERTIES OF CONCRETE

TESTING CONCRETE

23. Crushing Test.—The test usually made for concrete is a simple compressive test. Tests of cement, sand, and broken stone taken separately are described elsewhere. Concrete is tested in blocks. The size of the block to be tested is not definitely fixed by any standard, and depends on the size of the testing plant. In tests of simple compression, blocks of cubical form are generally used; while in tests of columns and in investigations regarding the modulus of elasticity of the material, rectangular prisms are used whose altitude is between five and eighteen times their width.

When crushing cubical blocks, it is always found that the larger the block the higher is its crushing strength per square inch. To approximate the conditions that actually exist in the work for which the concrete is intended, as large a block should be used as possible, because if a smaller block is taken the concrete will be stronger than indicated by the test. The size of the specimen, however, is limited by the size of the testing machine. The testing machine at the Watertown Arsenal, which is one of the largest testing machines in the world, has a capacity of 800,000 pounds. This machine will take very easily a cube 12 inches on a side. This gives 144 square inches area, on each of which the machine can exert a pressure of 5,556 pounds. However, most testing machines are not so large as this, and a cube 6 or 8 inches on a side is usually taken for testing purposes.

24. The concrete used to make sample blocks is generally mixed by hand. Sometimes, however, samples are taken directly from the work as the concrete is about to be deposited, and in this case, of course, the concrete may

be machine-mixed. The concrete when mixed by hand in the laboratory is made into a fairly wet mixture and put in a mold similar to that shown in Fig. 1. Great care must be exercised in making the concrete and in placing it in the mold. The mold is made of wood, usually white pine dressed on the side next to the concrete, and oiled on the inside so that the concrete will not stick to it in setting. The mold can be taken apart by unscrewing the four handles shown at *a*.

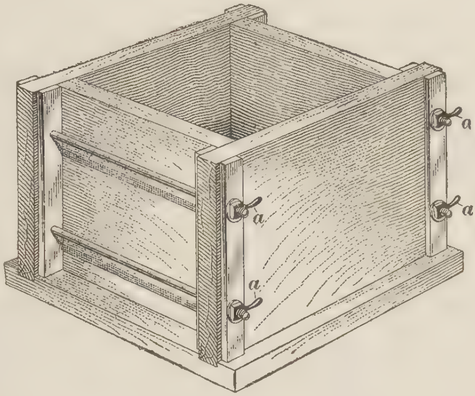


Fig. 1

The concrete is first mixed until it acquires a uniform color, and then it is deposited in the mold. A layer of about 4 inches is first put in and thoroughly tamped down. Another layer of 4 inches is then deposited, and the mass is again rammed, and so on until the mold is entirely full. The mass should then be of such consistency that water will be brought to the surface by ramming.

After the concrete has set for the required time and is ready to be tested, the two sides of the block on which the machine presses should be smoothed so that the machine will bear evenly all over. To do this, the cracks and irregularities on these two faces are usually filled in with plaster of Paris or neat cement applied in a thin coat and pressed between metal plates until it has set. Any little irregularities then remaining are scraped off with a knife.

25. Other Tests.—Other tests are made on concrete besides the compressive test. In reinforced-concrete work, beams and columns are sometimes, although seldom, tested to destruction. These are made with the same care that is

TABLE III

AVERAGE ULTIMATE STRENGTH OF CONCRETE MADE FROM PORTLAND CEMENT, SAND, AND CRUSHED STONE

Proportion of Ingredients			Tension Pounds per Square Inch				Compression Pounds per Square Inch				Shear Pounds per Square Inch			
Cement	Sand	Stone	7 Days	1 Month	3 Months	6 Months	7 Days	1 Month	3 Months	6 Months	7 Days	1 Month	3 Months	6 Months
1	2.0	4	160	210	240	250	1,600	2,150	2,400	2,500	200	269	300	313
1	2.5	5	143	195	225	235	1,430	1,950	2,250	2,350	179	244	281	294
1	3.0	6	125	180	210	220	1,250	1,800	2,100	2,200	156	225	263	275
1	3.5	7	110	166	196	208	1,100	1,660	1,960	2,080	138	208	245	260
1	4.0	8	98	152	182	195	980	1,520	1,820	1,950	123	190	228	244
1	4.5	9	85	140	169	184	850	1,400	1,690	1,840	106	175	211	230
1	5.0	10	75	126	155	172	750	1,260	1,550	1,720	94	158	194	215
1	5.5	11	65	112	142	160	650	1,120	1,420	1,600	81	140	178	200
1	6.0	12	60	100	130	150	600	1,000	1,300	1,500	75	125	163	188

exercised in making the compression pieces. They are tested by special machines made for the purpose, or by special appliances fitted to ordinary testing machines. They are sometimes roughly tested to destruction by loading them with pig iron at the place where the work is being carried on.

26. The adhesion with which concrete will stick to steel is also sometimes tested. The concrete is made into a block, and a steel rod of known dimensions is inserted in it. When the concrete has set, the block is put in the testing machine and the steel rod is drawn out. By knowing the number of square inches of steel in contact with the concrete, and the force necessary to draw out the rod, the adhesion per square inch may be found.

27. The strength of concrete in shear is a feature of considerable importance, especially in connection with reinforced-concrete work. So far, however, few reliable tests have been made on the subject.

28. Table of Ultimate Strengths of Concrete. Table III gives the average results of tests made on concrete. It has been compiled by W. Purves Taylor, engineer in charge of the municipal testing laboratory of Philadelphia, Pennsylvania. The figures represent average values obtained from six hundred experiments made on concrete properly mixed with good Portland cement.

The increase of strength of the concrete with age, which can also be seen by reference to Table I, will be noted. The columns giving values of concrete in tension are of comparatively little value to the engineer. The table gives values for concrete made with broken stone. Concrete made with gravel is about 75 per cent. as strong, and concrete made with cinders is about 65 per cent. as strong. Table III is recommended for use in all general work with concrete made with high-grade Portland cement.

SETTING

29. Setting in Freezing Weather.—It is safe to say that concrete made with natural cement is seriously injured and its strength seriously impaired if laid when the thermometer is much below the freezing point. On the other hand, Portland-cement concrete does not seem to be affected so much by the cold. Chilled concrete, it is well known, sets much more slowly than warm concrete. If the temperature is so low that the water in the concrete has time to freeze it usually happens that there is not enough water left on the surface to form a chemical union with the remainder of the cement. The frost, on the other hand, will hold the concrete solid. On the first warm day of spring, the frost thaws and the concrete becomes somewhat crumbly and soft, until it absorbs enough of the water liberated to acquire its final hard set. After this, the concrete will be as solid as ever, except perhaps the outside layer, about $\frac{1}{8}$ inch thick, which may have peeled off. The danger is that the contractor or the person doing the work during the winter may load the concrete with a bridge or whatever other load it is intended to withstand, and when the spring thaw comes, but before the concrete has had time to sufficiently set, the load may be greater than the concrete can stand. If concrete is laid in freezing weather, it should not be loaded until after it thaws.

30. Many methods are devised to prevent concrete from freezing. Some engineers add a little lime to the mixture; the beneficial effect of this is perhaps doubtful. Other engineers make the concrete a little richer in cement than usual; this probably has some good effects. The best method of laying concrete in freezing weather is to heat some or all of the ingredients. If salt is added to the water in which the concrete is made, it will lower the freezing point of the mixture, but may also stain it and have a slight chemical effect on the cement. After the concrete is laid, it should be covered with tarpaulin or clean straw, but not

with manure, as is often done, because the acids in the manure may disintegrate the concrete.

31. Contraction of Concrete on Setting.—Another peculiarity of concrete is that when it sets in the air it contracts slightly, and when it sets in water, it expands. This change does not occur in the body of the sand or stone, but in the cement between the grains of sand. Concrete will, therefore, shrink less than pure cement. The shrinkage is apt to cause cracks in the finished work, especially when the concrete is not reinforced by steel. To prevent the occurrence of these cracks in small work, the concrete is kept moist for some time after it is placed. As concrete expands when it sets in water and contracts when it sets in air, there is some intermediate point that can be approximated by sprinkling with water, so that the concrete will neither expand nor contract. Therefore, when laying concrete pavements and the like, it is customary to keep the work well sprinkled for several days after it is placed. Sometimes, in large work, concrete is laid with **planes of least resistance,** the object of which is not to prevent the concrete from cracking, but to make it crack in such a manner that it will neither impair the strength of the structure nor look unsightly. These planes of least resistance are usually made by inserting pieces of tar paper in the concrete, and thus dividing the work. The tar paper should come close to the surface on all sides; and if it is properly inserted, it will be found that the work will crack in a plane with the paper. An attempt is sometimes made in dam work and other watertight structures to fill the expansion cracks with asphaltic cement, so as to make an expansion joint. Under such conditions, the best way is to make the expansion joints 2 or 3 inches wide, at least, and then fill them with the asphaltic cement.

32. Setting in Sea-Water.—Concrete is not impervious to sulphuric acid and soluble sulphates. Concrete laid under salt water will be slowly disintegrated by the action of the sulphates in the water. The same is true of concrete laid

near gypsum or coal mines, where the sulphates in the drainage water will wash against it. Concrete laid under seawater, or in any other place where it is subject to the action of sulphates, should be made with the cement low in aluminum and lime. Pozzuolana cement makes a better concrete for use under water than does Portland cement.

MIXING OF CONCRETE

HAND MIXING

33. In making concrete, the proper mixing of the ingredients is of the utmost importance. Different structures can be made from the same materials in the same proportions, but, if the ingredients are mixed in different manners, one structure may be entirely satisfactory while another may fail. The mixing of the ingredients to make concrete may be performed either by hand or by machine. The two methods, if properly applied, give about the same grade of concrete. When only a little concrete is required, it is generally found cheaper to mix it by hand than by machinery; but when a large amount of concrete is required at one place, it is generally cheaper to use a machine.

34. In preparing concrete by hand, the material should be worked on a platform of boards, with sides about 10 inches high, battened on the back, and laid on the ground near the work. The platform is necessary in order to keep the concrete off the ground, so that neither loam nor clay may contaminate it; the effect of such contamination is a loss of strength, as the clay adheres to the stone and prevents close contact with the mortar. The measured quantities of sand and cement should first be thoroughly mixed by turning them over while dry, at least twice, so that there will not be different proportions of sand to cement in different parts of the heap. It can generally be told by the uniformity of color and absence of streaky appearance when the mixing of the sand and cement has been properly accomplished.

This mixture is then wetted with water, and is shoveled over again until it is of a uniformly pasty consistency; it is then spread out on the platform in a layer about 6 inches thick, and the broken stone, which has been previously wetted, is distributed evenly on it. The mixture is then turned over with a shovel twice for final mixing, when the concrete is ready to be put in place. The concrete should be laid as soon as mixed, before the cement has had time to set. The mixing box should be placed convenient to the work, and at the same time as near as possible to the place where the ingredients are stored. Care must be taken to see that wheelbarrows, dump carts, or whatever conveyances for the materials are used have easy access to the mixing box. If possible, the mixing box should be so placed that the materials will move continually down hill, as it is easier and cheaper to lower materials than to raise them.

MACHINE MIXING

35. Mixing machines may be divided into two general classes; namely, **batch mixers** and **continuous mixers.** Batch mixers, as the name implies, take a certain quantity or batch of ingredients, mix them, and deliver them. The machine is then ready for another batch. A continuous mixer, on the other hand, is one to which the unmixed materials are continuously fed, and which is continuously delivering mixed concrete.

BATCH MIXERS

36. The Cube Mixer.—One of the oldest and best-known forms of batch mixers is the **cube mixer.** It consists essentially of a cubical steel box revolving on a hollow horizontal shaft passing through diagonally opposite corners of the box. This box is charged and discharged through a trap door, placed near one of its corners. In operation, the mixer is revolved until the corner where the door is comes uppermost; then the door is opened, and the ingredients are

dumped through a chute or hopper into the mixer. The door is then closed, and the box is revolved; after a few turns to mix the materials, the necessary water is introduced

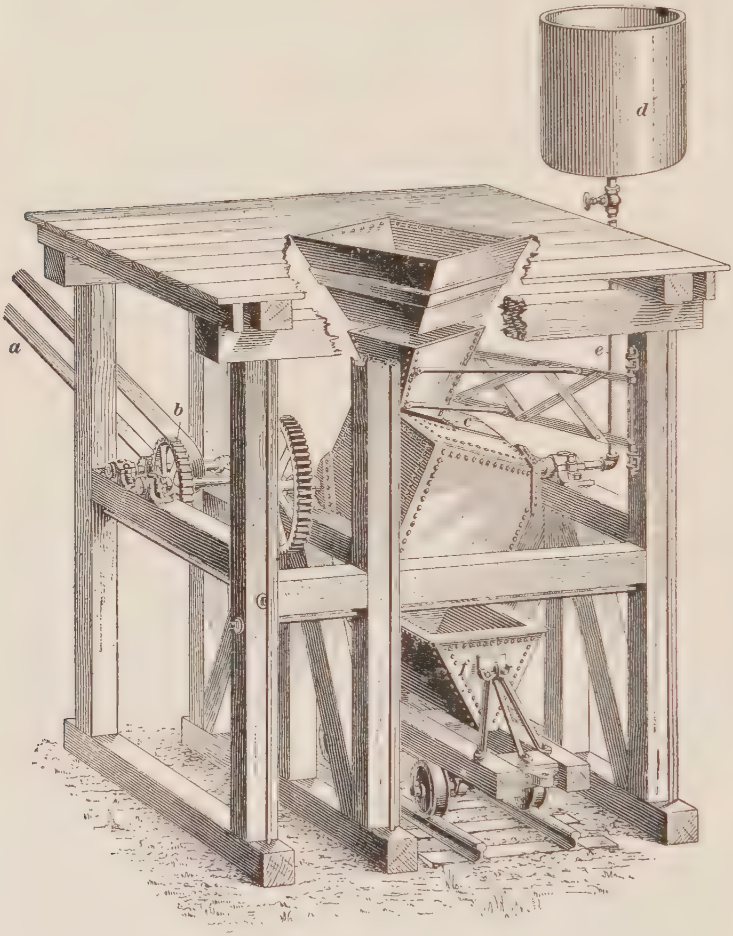


FIG. 2

through the hollow shaft, and then the revolving is continued for some time, after which the box is stopped, with the door at the lowest corner. The door is opened, and the mixed concrete is dumped into a car or other receptacle and taken away.

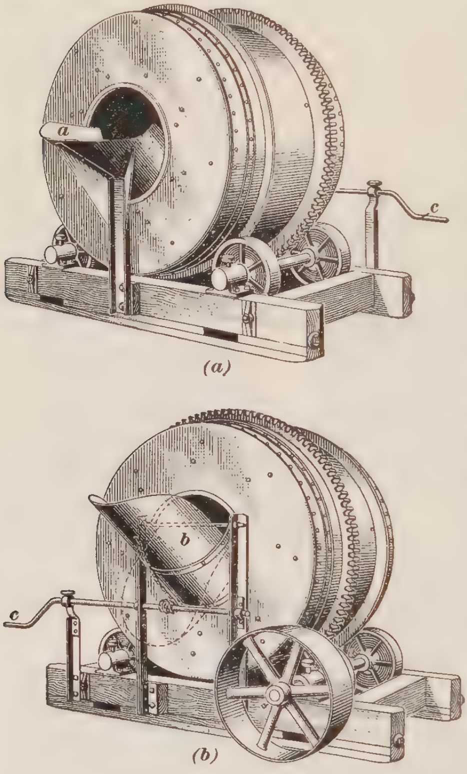
The usual size of the cube or box is 4 feet on each edge. This size has a nominal capacity of 1 cubic yard of rammed concrete. The ingredients loose should fill the mixer about half full; with a larger charge, the concrete may not be thoroughly mixed. In using this mixer, care must be taken to keep it clean, and to prevent accumulations of particles of mortar in the corners or on projections inside the cube. This can be done by pounding strongly on the outside of the cube with a wooden mallet or maul as each batch is dumped out; the remaining clinging particles of mortar are thus detached before they have time to harden in place.

A cube mixer is usually mounted on a substantial framework of timber, as illustrated in Fig. 2, in which is shown a complete form of cube mixer made by Thos. Carlin's Sons Company, of Allegheny, Pennsylvania. The cube is driven by a belt *a*; gears at *b* serve to reduce the speed of rotation. The cube is shown in the position it occupies when being filled. The trap door for filling and discharging is shown open and swung back at *c*. The hopper above the cube is made in two parts: the lower part is hung from one of the posts of the platform, as shown. When the cube is revolving, this lower part can be swung out of the way. At *d* is shown the water tank, which feeds water into the cube through the pipe *e*. Below the cube, at *f*, is shown a car into which the finished concrete is dumped.

If the nature of the work is such that the ingredients can easily be taken to the platform above the cube, this machine will make an economical arrangement, because after the materials are once shoveled into the hopper above they do not have to be handled again until the concrete is deposited in the conveying car.

37. The capacity of a cube mixer varies according to the volume of the charge and the time interval between successive batches. Various sizes of charges and various rates of speed have been used by different engineers, ranging from a charge sufficient to produce 18 cubic feet of rammed concrete to one producing 27 cubic feet, and from a speed of six to one

of fourteen turns per minute. The number of charges per hour will vary according to the speed of revolution of the mixer; this should not be great enough to prevent by centrifugal force the free action of gravity on the enclosed mass causing the ingredients to tumble or roll over as the box revolves.



(a)
(b)
FIG. 3

The following conditions have been found to produce excellent results with a 4-foot cube mixer: Each batch is mixed $1\frac{1}{2}$ minutes at a speed of ten revolutions per minute, making a total of fifteen turns per batch. The time allowed for dumping, cleaning, and charging the mixer, and for temporary stoppages of the work, is about $3\frac{1}{2}$ minutes, making the average time interval between successive batches 5 minutes, which is at the rate of twelve batches per hour. Allowing only $\frac{7}{8}$ cubic yard of rammed concrete per batch, this gives for a 10-hour day an output of 105 cubic yards.

38. The Ransome Mixer.—Another form of batch mixer is the **Ransome mixer,** illustrated in two views in Fig. 3. It is made by the Ransome Concrete Machinery Company, of New York. This machine consists essentially

of a hollow cylindrical drum, mounted on a horizontal axis, and having a circular opening in each end. The mixer is charged through one of these openings, shown at (*a*). Through the other opening, shown at (*b*), is a chute that receives the mixed concrete from the drum and delivers it to a wheelbarrow or other receptacle used for removing the concrete. Inside the drum are several blades or wings, arranged in such a manner as to deflect the material from side to side as the drum revolves. The drum is mounted on four rollers, and is supported on a truck, which is either made stationary or mounted on wheels, as required. The mixer is turned by power applied to the rim of the drum, either from an engine mounted on the same frame or from a belt or chain. The process of mixing is as follows: The required quantity of water for a batch is first placed in the mixer; then, the cement, sand and broken stone, previously measured, are dumped in as the drum revolves.

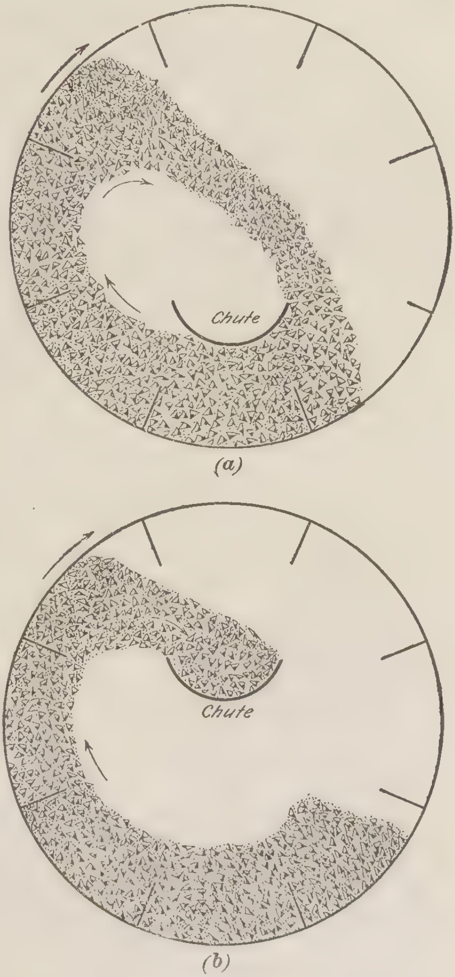


Fig. 4

The chute *b*, through which the mixer discharges, is kept in the position shown in Fig. 3 (*b*) while the concrete is being mixed. When it is time to empty the mixer, the handle *c* is turned, which, by connection to the chains shown, tips up the chute *b*, as illustrated by dotted lines in the figure. Meanwhile, the drum is revolving and lifting masses of concrete up one side in the direction of its motion; these masses finally fall by the action of gravity to the bottom of the drum again.

The path followed by the concrete in the mixer is shown diagrammatically in Fig. 4 (*a*). The blades are represented by short radial lines. When the chute is tipped down, as shown by dotted lines in Fig. 3 (*b*), the upper end intercepts the flow of concrete, as shown in Fig. 4 (*b*), and the mass slides down the chute and out of the drum.

This form of mixer is made in several sizes, having a capacity up to 30 cubic yards per hour.

39. The Gilbreth Rotary Mixer.—A mixer somewhat like the Ransome mixer is illustrated in Fig. 5. It is known as the **Gilbreth rotary,** and is made by the United Concrete Machinery Company, of New York. This mixer differs from the Ransome in that, instead of having a chute for feeding or discharging the concrete, it is made so that a wheelbarrow can be put directly inside the drum. The mixer can be fed or discharged from either side. The inside is fitted with heavy steel vanes or shovels that carry about 1 cubic foot of concrete apiece to the top of the mixer, and there let it drop into the mass below or into the wheelbarrow. This mixer will produce from 15 to 30 cubic yards of concrete per hour.

40. The Smith Mixer.—This machine, as furnished by the Contractors Supply and Equipment Company, of Chicago, Illinois, is illustrated in Fig. 6. It is self-contained and portable, the mixer, engine, and boiler being mounted together on a truck, which is supported on wheels. The machine consists of a drum of double conical form, supported and guided by a frame, which can be tilted at will

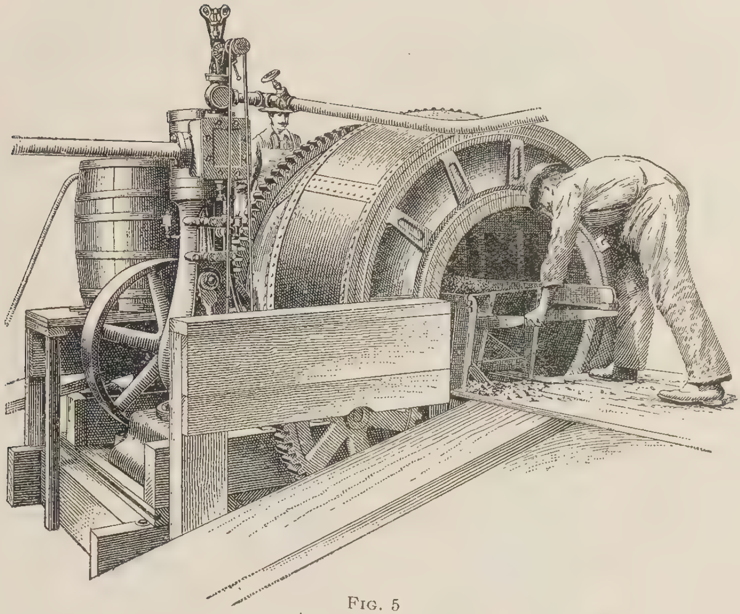


FIG. 5

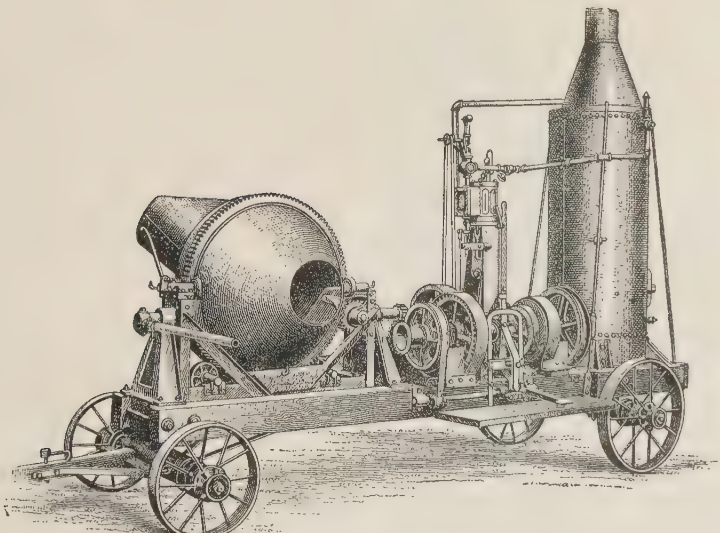


FIG. 6

while the drum is revolving. The two conical sides of the mixer are connected at their bases to a central ring, which is provided with gear-teeth for revolving, and machined surfaces for guiding the drum. The conical sides are truncated, having circular openings at their ends for the reception and discharge of the material. The ingredients are fed in at one end of the drum, and after the required number of revolutions the mixed concrete is discharged at the other end by tilting the drum while it is running at full speed. The interior of the drum is provided with blades arranged so as to insure thorough mixing. The capacity of the mixer is from 10 to 35 cubic yards of concrete per hour, according to the size of the machine.

41. The International Concrete Mixer.—Fig. 7 illustrates the **International concrete-manufacturing apparatus,** made by the United Concrete Machinery Company, of New York. The mixer, which is very much like the Ransome mixer, is shown at *a*. It is not the mixer itself, however, that deserves attention, so much as the remainder of the device. This apparatus is designed to handle the concrete from the time the ingredients are proportioned to the time the finished concrete is dumped into the car ready to be hauled to the work and put in place. The measured ingredients are dropped into the car *b*. The car is then hauled up the plane by the engine shown at *c*. As the car ascends, the front wheels, which are of narrower gauge than the hind wheels, run on a track on the horizontal girt, while the hind wheels on the outside tracks continue up the plane. This arrangement brings the car into the position shown by dotted lines in the figure, and the contents of the car slide out into the hopper *d*, and thence into the mixer itself. The engine runs the mixer continuously. The hoist that hauls up the car is operated by the lever *e*. This lever is thrown over, operating a clutch driven by the two miter wheels shown at *f*, when it is desired to elevate the car. When the car reaches the position shown by dotted lines, the clutch is thrown out, but the car is held

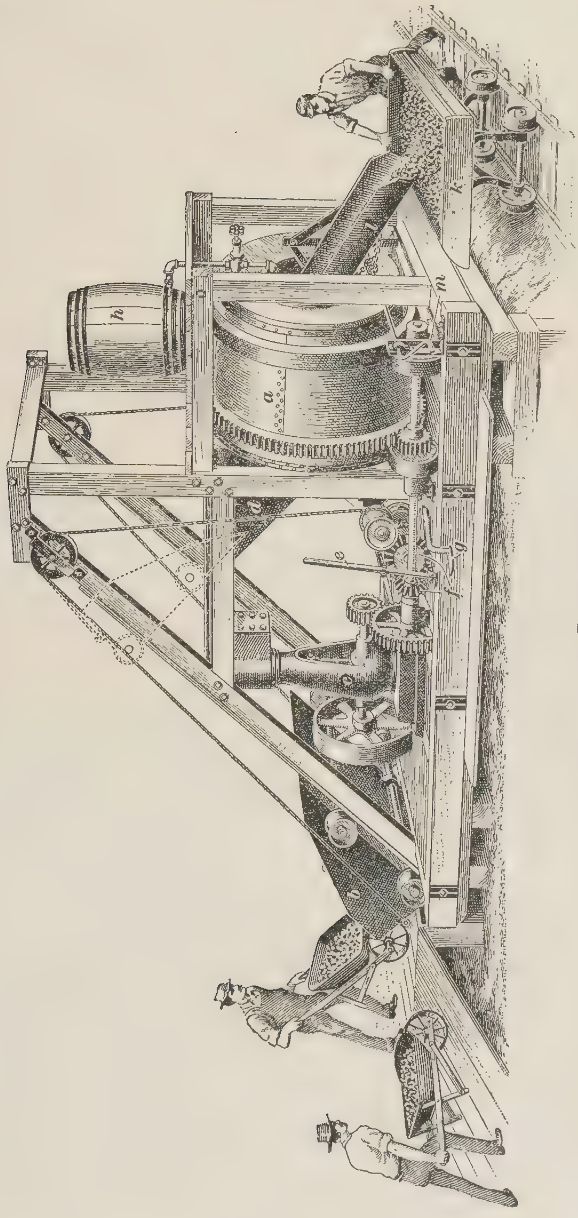


Fig. 7

by the operator putting his foot on the lever *g*, which operates a band brake on the hoisting drum, not shown in this illustration. When it is desired to lower the car, the operator slowly takes his foot off the lever *g*, and the car descends. To keep the car clean, so that it will dump quickly, the water is not added therein but goes direct to the mixer from a barrel *h*. The man that receives the mixed concrete in the car *k* tips the chute *l* by means of the handle *m*, as explained in connection with the Ransome mixer.

CONTINUOUS MIXERS

42. Principle of the Continuous Mixer.—The fundamental idea of the operation of a continuous concrete mixer is to feed into one end of the machine a steady stream of raw material and turn out mixed concrete at the other end. Theoretically, this is an ideal arrangement, but in practice it is difficult to maintain uniformity in the grade of concrete turned out without the aid of some special measuring apparatus. If the stream of cement, sand, and stone, together with the necessary water, is fed into the mixer in the right proportions, the resulting product will be a good grade of concrete. If, however, there are variations in the supply of the ingredients, there will be corresponding variations in the resulting concrete, which at one time may be rich in cement and at another time deficient in the same ingredient. If the ingredients are measured out in the proper proportions and fed into the mixer in the proper manner, a good continuous mixer will turn out as good a quality of concrete as is obtained from batch mixers. Manufacturers of continuous mixers have realized the necessity of such uniform distribution of the ingredients, and some of the leading forms of continuous mixers are provided with measuring and feeding devices to accomplish this purpose.

43. The Drăke Mixer.—One of the best-known forms of continuous-mixing machines is the **Drake mixer,** made by the Drake Standard Machine Company, of Chicago, Illinois. This mixer consists of an open trough, fitted with

a longitudinal shaft, to which are fastened blades or paddles set at an inclination, so that they will not only mix the ingredients, but also feed the mixture toward the discharge end. The mixer is set so that the shaft revolves on a horizontal axis. The ingredients are deposited by wheelbarrows or from a measuring box at the upper or feeding end of the trough. The straight blade or knife at that end cuts or stirs the mass, which is then turned over by the adjacent curved blade or scoop, and advanced to the next knife, where it is again cut and then turned over by the next scoop, and passed on. This process is repeated until the end of the trough is reached, when the material is pushed out and falls into a receptacle as mixed concrete. The ingredients are mixed dry for about one-half of the length of the trough, and then a spray from water pipes deposits water on the material as it is cut and turned over. Various sizes are made, each mixer having a capacity, as stated by the makers, of from 75 to 200 cubic yards of concrete per day, according to the size.

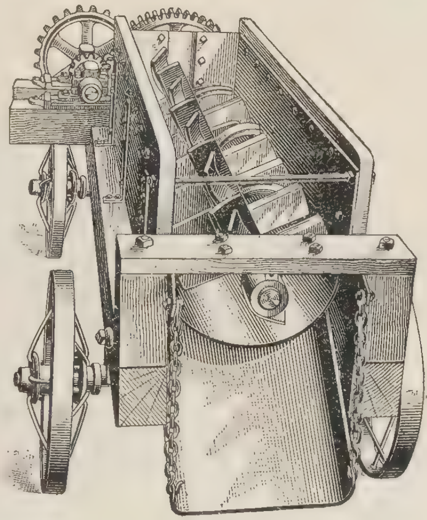


Fig. 8

Fig. 8 shows a Drake mixer of usual design. The ingredients are dumped in at the far end, and when they are entirely mixed are delivered down the chute. The company makes mixers in various styles. The more elaborate ones have belt conveyers attached to the machine to deliver the ingredients and take away the finished concrete. Some are built direct-connected to an engine, and others are made to be driven with a belt or chain, as illustrated. Some machines are portable, and others are made to be kept in one place;

but they all have the characteristic horizontal shaft with paddles on it.

44. The Cockburn Mixer.—Another form of continuous mixer is the **Cockburn mixer,** which is illustrated in Fig. 9. It is made by the Cockburn Barrow & Machine Company, of Jersey City, New Jersey. This machine consists essentially of a long box of square cross-section, mounted on a substantial iron frame or truck and revolving on a longitudinal axis on friction rollers, as shown. The axis of the box is inclined slightly to the horizontal, and the box is revolved by spur gearing, the power being applied at the upper end. The ingredients are fed in through a hopper

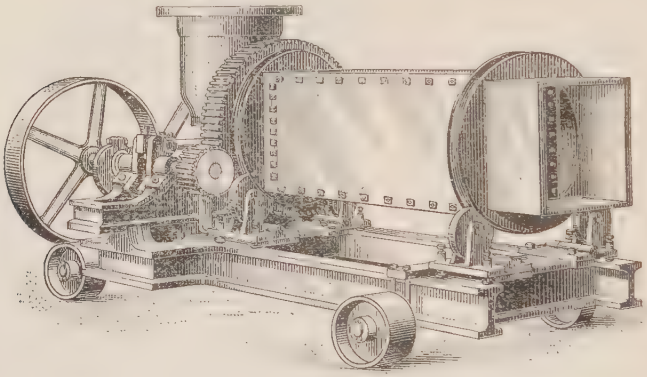


Fig. 9

or chute at the upper end, and, as the mixer revolves, the material moves by gravity toward the lower end, becoming well mixed by several turnings in its passage, and being finally discharged in the form of concrete. This is one of the oldest forms of continuous mixers now made. It has been used with satisfactory results on concrete work of considerable magnitude. The volume of its output depends largely on the rapidity with which the ingredients are fed into the mixer. The capacity of the machine is stated by its makers to be from 150 to 250 cubic yards of concrete per day.

45. The Portable Gravity Mixer.—The **portable gravity mixer** is made by the United Concrete Machinery

Company, of New York. A view of one of these machines is shown in Fig. 10. The mixer consists of a long inclined box or chute, surmounted by a hopper for receiving the materials. This box contains, at intervals in its length, a number of steel pins and deflectors, which interfere with the free passage of the ingredients and cause them to mix. Water is fed through a spray pipe about midway of the length of the mixer, this arrangement allowing the materials

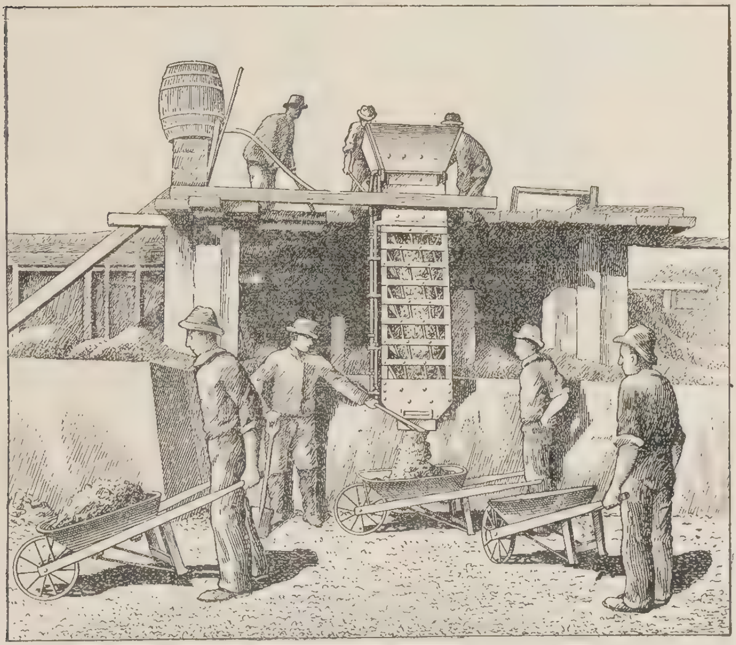


FIG. 10

to be mixed dry in the upper half and wet in the lower half. There is also another spray pipe at the top of the mixer, so that if desired the concrete can be mixed wet from the very beginning.

In operation, the proper quantities of stone, sand, and cement are spread on the platform in successive layers, the stone being at the bottom; they are then shoveled into the hopper of the machine, whence they slide down the chute

being diverted from side to side by the deflectors and the steel pins.

Fig. 11 shows one of these mixers in detail. It will be noted that there is a row of deflecting rods at the end of the

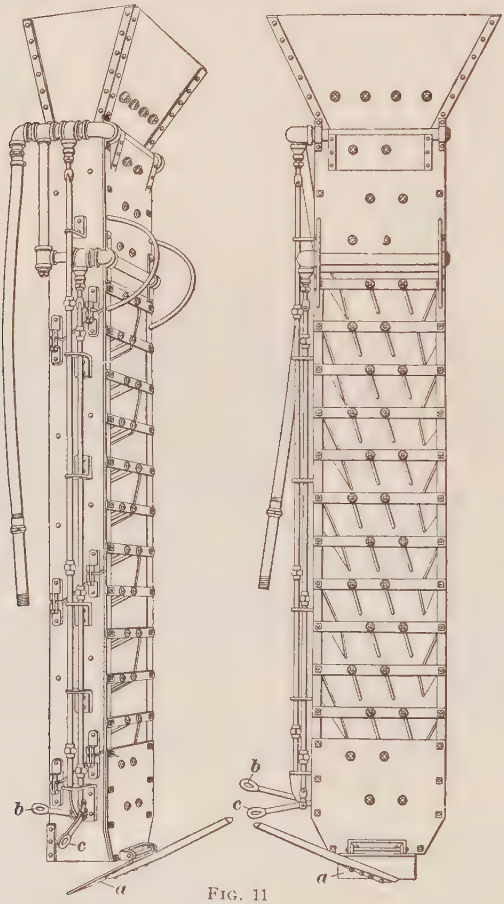


Fig. 11

receiving hopper, which are placed closer together than anywhere else in the machine. This is to make sure that any piece of broken stone not too big for the bars in the hopper will not clog the mixer anywhere else. The bottom of the mixer is provided with a door a, which the man at that end

closes when there is no car underneath. The same man has under his control the water, which is added by handling the levers *b* and *c*; these levers govern the flow of water at the top and at the middle of the mixer.

This type of mixer, in which the work is done by gravity, seems to be well suited for concrete work in places where steam or other power is not readily available. On account of its portability and ease of operation, this mixer should be well adapted for concrete work around mines and in mountainous regions where the cost of transportation would prohibit the use of heavy machinery for this purpose.

THE GILBRETH MEASURER AND FEEDER

46. A machine of considerable importance in concrete mixing, especially in continuous mixing, is the **Gilbreth accurate measurer and feeder,** made by the United Concrete Machinery Company, of New York. In works of great magnitude, this machine will be found economical. Cement, sand, and broken stone are dumped into three storage bins. The machine controls the supply of materials from each bin in such a manner that they come out in the desired proportions. The materials are delivered either to a hopper or directly to a mixing machine, as may be required.

Fig. 12 shows a view of this measurer. At *a*, *a*, *a* are seen the bottoms of the three storage bins, which are built hopper-shaped. In one of these bins is stored cement, in another sand, and in the third, broken stone. These bins, as already stated, are hopper-shaped, but have no flat bottoms. They are closed at the bottom by the cylinder *c*, which is of sufficient diameter to act as a practically flat bottom to the bin. In the front of each hopper there is a door *b* that is adjustable by a weight, as shown in the figure. The cylinder *c* is turned by a man at the handle *d*. As the cylinder revolves, it carries with it a layer of cement, sand, and broken stone. The amount of each material carried forwards, or, in other words, the depth of each material on the cylinder, is controlled by adjusting the gate at *b*. As

the wheel revolves, the materials finally slide off into a conveyer or directly into the mixer. The faster the operator revolves the cylinder, the more materials he will discharge, but they will always be discharged in the same proportion.

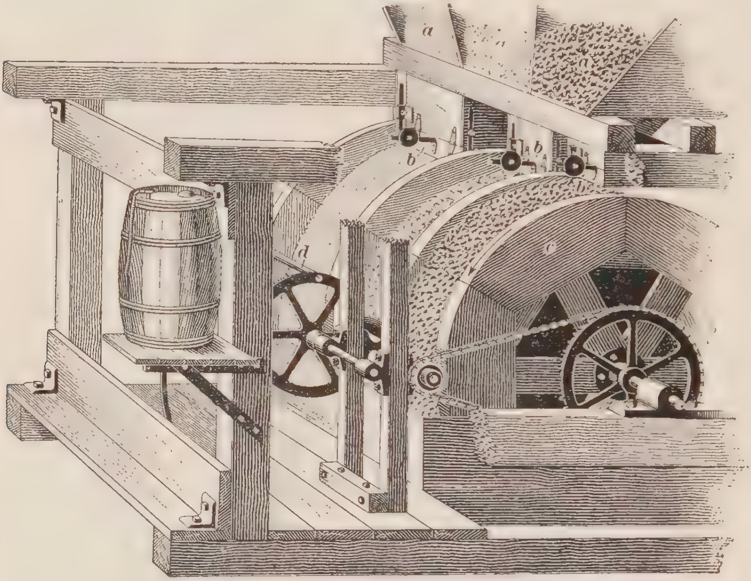


FIG. 12

The operator has only to attend to the signal of the man at the mixer as to when and how fast to feed the materials. If this feeder can be used in connection with a gravity mixer, no steam power is required, and the concrete can be made very cheaply.

REMARKS ON CONCRETE MIXING

47. The question as to what method will be used in mixing concrete is of importance. When the volume of concrete to be mixed is sufficiently large to justify the expense of installation, a mixing machine should be used. While equally good results can be obtained by hand and by machine mixing, the work done by hand is likely to be

uneven in quality, and some batches will be more thoroughly mixed than others. On the other hand, the work done by a machine is even and regular, and machine-mixed concrete is usually of more uniform quality than that mixed by hand. When the question of cost is considered, it is found that the advantage is decidedly in favor of machine mixing, provided that there is sufficient work to warrant the initial expense of buying and setting up a machine.

With regard to the selection between machine mixers, there is little definite that can be said. The mixers that have been described are simply representative types, and many concrete-mixer companies make mixers in almost all the styles mentioned. If the directions of the manufacturers are followed, any of the mixers described will make good concrete. The engineer, in choosing a mixer, should see that it is of good workmanship and good material. It must be heavy enough to stand the wear that a machine of this character is sure to receive. Whether the mixer can be placed to conveniently receive the cement and aggregates, and whether it will deliver the concrete in a convenient place, are matters that should be carefully considered. The amount of power required to run the machine and the number of men required to operate it are also of importance.

PLAIN-CONCRETE CONSTRUCTION

48. Conveying the Concrete.—Immediately after the concrete has been mixed, it is placed on the work. Usually, it is carried in carts or wheelbarrows to the forms. Sometimes, it can be deposited directly from the mixer by a conveyer or other appliance. There are no fixed rules for conveying concrete from the mixing place to the work; this is a matter that depends to a great extent on local conditions. The following points, however, should be observed.

The concrete must be deposited before it has time to set. The mixing should be done as near the place where the concrete is to be deposited as possible, so as to avoid having to carry it a long distance. If possible, the mixing should be

done at a higher level than that at which the concrete is to be placed, to avoid lifting materials; in fact, the concrete may sometimes be slid to place down a trough. If concrete is to be hauled by laborers in wheelbarrows or carts, the engineer should see that the line of men going to the work and that of the men going back to the mixing place for more concrete do not get in each other's way, and that there are no delays at either end of the line to keep the men waiting. The cheap laying of concrete depends more on the skill with which the engineer handles his men in hauling the material to the molds than on anything else.

Fig. 13

Fig. 14

49. Laying the Concrete.—The concrete is usually deposited in a layer from 4 to 6 inches thick, and is then rammed in place. If the concrete is dropped a considerable distance to the work, or even slid rapidly down a trough, the ingredients are likely to separate, especially if the concrete is very wet. The concrete should, therefore, be shoveled over before being rammed in place. In large work, when there is a considerable drop from the place of mixing to the place of deposit, the concrete is usually lowered in a bucket.

In very wet concrete, the mixture will flow readily into place, and will require only a little poking and stirring with a spade to liberate all the air bubbles. In concrete, of the usual consistency, however, which is not absolutely liquid, considerable ramming is necessary to make the work compact.

50. Rammers.—Various kinds of rammers are used; and in small work, many improvised tools, as pick handles, are employed. Figs. 13 and 14 show two rammers that

are in common use today. The form shown in Fig. 13 is used to get into smaller places and where harder ramming must be done. The form shown in Fig. 14 is the more common form for ordinary work. Both these rammers have straight ash handles, so that a man may stand erect while he works. Sometimes, on large work or on dry concrete, as in concrete-block work, a machine rammer is used, operated by compressed air or steam. It works very much on the same principle as a pneumatic hammer. The machine concrete rammer has, however, not come into general use.

MOLDS

51. General Principles.—It is usually necessary, when placing concrete, to confine it in forms or molds. This is essential on account of its tendency to run and spread while still wet. The molds are usually of wood. Sometimes, below ground, concrete is simply put into the excavation and confined by the earth walls. This is not likely to make as firm concrete as when wooden forms are used.

In building concrete molds, there are a few fundamental principles that it is necessary to learn; outside of these principles, the details of the work are mainly a matter of experience and judgment. In the first place, the timber used for making the molds should be injured as little as possible, as it may be taken down and used over again either for more forms or for other purposes. On account of their temporary character, no more time or expense should be spent in making the molds than is absolutely necessary. It is injurious to the concrete to harden in a mold that is unsteady, as this interferes with the setting. For this reason, molds should, except for very light work, be built perfectly solid, usually out of 2-inch plank. Most authorities claim that molds should be comparatively waterproof, but it seems that it is sufficient to have the mold just tight enough to prevent the concrete from running through the cracks between the timbers.

It is customary to use dressed lumber in work above ground. When undressed stuff is used, the rough surface of the timber is cast plainly on the surface of the concrete, which presents an unsightly appearance. For the same reason, it is bad practice to use stuff that will warp when wet. The best molds for concrete are made of a good grade of white pine.

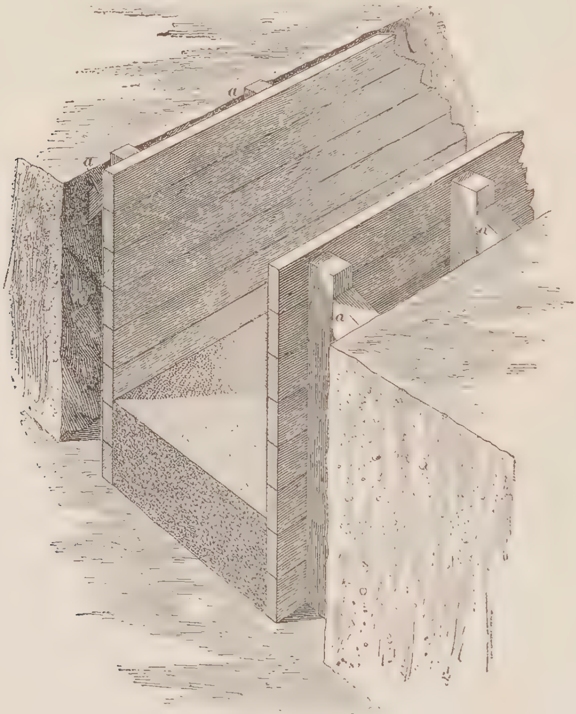


Fig. 15

52. Sometimes, concrete has a tendency to stick to the molds. When these are removed, therefore, some of the surface of the concrete comes off, and the lumber has to be scraped before it can be used again. When this adherence takes place, the work has a rough, unpleasing appearance, and has to be patched up with a trowel, or *made good*, as this operation is called. There are several ways of preventing the concrete

from sticking to the molds. If the lumber is planed, the concrete is much less likely to stick. Sometimes, the inside of the mold is painted with a heavy mineral oil (not animal oil, as the fatty acids it contains attack the concrete). Sometimes the molds are lined with building paper. In the false work under arches, building paper of a strong and durable character is generally used. Tin or sheet iron is occasionally employed. Zinc should never be used, as it seems to attack the cement chemically.

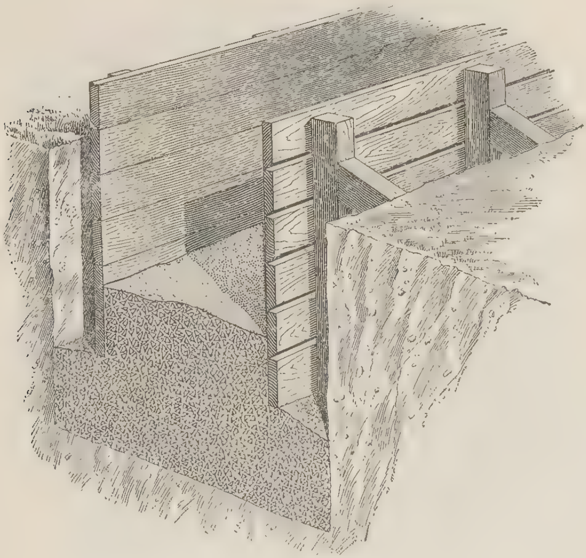


Fig. 16

53. Examples of Mold Construction.—One of the simplest constructions for a mold form for concrete is shown in Fig. 15. The form is composed of 2-inch plank supported by 3″ × 4″ posts. These posts are, in turn, braced by side braces, as shown at *a*. After the concrete has set for several days, the forms are removed and used over again, but the concrete should not be loaded for at least 1 week after it is in place. The illustration shows the concrete being placed in a ditch. The braces, however, may be made longer, and stood against the ground.

54. Fig. 16 shows a mold similar to that shown in Fig. 15, but with one or two improvements. In this form, the wall has a footing course that is wider than the remainder of the wall. To do this, the side boards are raised a little off the bottom, say about 18 inches. The concrete, as it is poured into the form, runs out at the bottom to a certain depth, and is tamped down with rammers from the outside. It will be noted, however, that to withdraw the posts

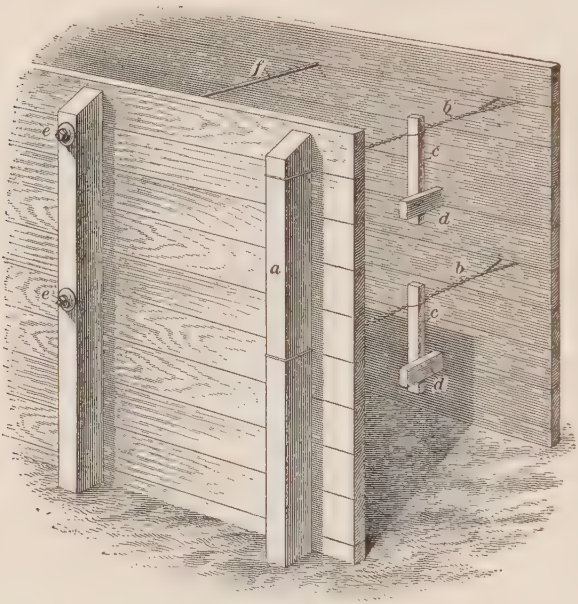


Fig. 17

they must be tapered and greased, and should be taken out as soon as the concrete is set enough to stand alone. The holes left by the posts may then be filled with concrete or earth, as desired. However, when lumber is cheap, it is better not to drive the posts any farther into the ground than necessary, and, after the concrete has set, to cut them off as short as possible.

Another feature of the form illustrated in Fig. 16 is that each of the side boards is chamfered on its lower edge. The

advantage of this is that if the planks swell, on account of their being wet, each plank can slide past its neighbor, and none of them will warp or spring. This refinement is, however, not often used in practice, except in reinforced-concrete work.

55. A form of mold used in large and heavy work is shown in Fig. 17. The planking is held in place by the posts *a*. These posts are not braced, but are tied through the mold in pairs. The ties *b* are made of wire. They are

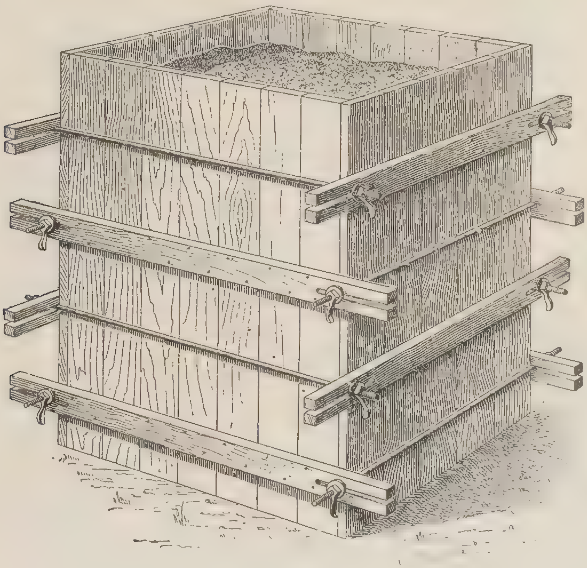


FIG. 18

simply looped around each post and run through a crack in the planking. In order to adjust the distance between the posts, the two wires are twisted together, as shown. They are then held by a stick *c*. This stick is kept vertical by a block *d*. The posts can be pulled closer together or can be let spring farther apart. The sticks and blocks are left in the concrete, but as they are so small compared with the entire mass, they do not seriously impair its strength. After the concrete has set, the foreman cuts with a hatchet the

wires on the back of the upright posts. The forms can then be removed. If desired, the ends of the wires left may then be cut off close to the concrete. In some cases rods are used instead of wires as shown at *f*, the ends being held by nuts and washers *e*, *e*, Fig. 17.

56. Another method of building forms, particularly for piers and foundations not over about 8 feet square, is shown in Fig. 18. This construction is very easy to remove, and the same timber can be used many times. It is not, however, adapted for very large work, because the outside braces, to be made strong enough, must be made too heavy and cumbersome.

MISCELLANEOUS CONSTRUCTIVE DETAILS

57. Finishing Work.—While it is not the purpose of this Section to discuss ornate concrete or concrete molding in round or curved shapes, as the matter of making forms for such figures hardly concerns the engineer, there are certain styles of finish to concrete work that are sometimes used, no matter for what purpose the structure is intended. It is often considered desirable, especially when building reinforced-concrete work, to chamfer the edges of all square columns and piers. This chamfering prevents the edges from being chipped off. It is effected by putting a triangular batten in the mold, as shown in Fig. 19.

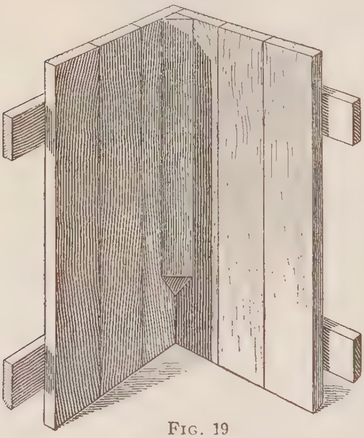
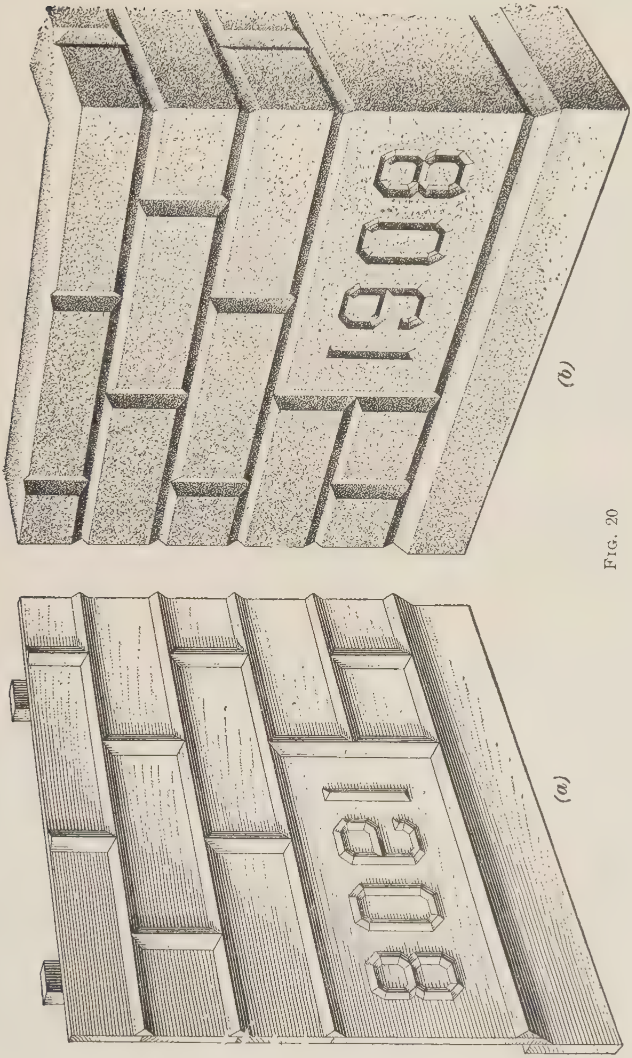


Fig. 19

Fig. 20 shows one method of making false joints in concrete work. The mold shown in view (*a*) has battens tacked on its face which produce the grooves shown in view (*b*). These grooves are supposed to represent joints in the stone that the concrete is intended to imitate. The tendency of the



(a) (b)

Fig. 20

day, however, is not to try to make concrete imitate any other material. It will be noted in the figure that any letters that it is desired to imprint on the concrete must be put backwards on the mold.

Sometimes, after the mold is removed, the face of the concrete is picked, usually with a small stone pick, to represent natural stone. This operation will be much facilitated if commenced as soon as the concrete has obtained its initial set, and before it becomes very hard.

Fig. 21

Another method of finishing concrete is by what is termed **washing.** The mold is taken off the concrete as soon as the mass is strong enough to stand by itself, and the surface is washed with water from a hose. This takes off the top coat, which is usually composed of cement and sand, and discloses the stones set in mortar, as shown in Fig. 21.

58. Joining Old and New Work.—Fresh concrete does not strongly adhere to concrete that has set. When concrete is being laid, there is always danger when part of the work has set, over night for instance, that the remainder of the work will not stick to the old part. Old and new work must be joined with some precaution. The old work should be clean; and, if it has been laid for some months, it should be washed on the surface to which the new concrete is to adhere with a mixture of cement and water. The first layer of the new concrete should contain rather more water than is customary to use for ordinary work. If the old work has been placed for some time, say 6 months or more, and if the surface has been finished smooth, it is well to roughen it up with a pick before the new concrete is put on. Concrete work to which it is intended to make additions should never be left smooth. Some engineers embed short pieces of iron

or steel projecting up from the old work; the new work grips to this metal, which thus forms a tie. If the old work is left very rough and irregular, it will probably not be necessary, except in extreme cases, to use metal ties.

59. Laying Concrete Under Water.—In general, concrete is laid in the same way under water as in the air. There is only one precaution to be taken, and that is to prevent the cement from being washed away before it has set. There is no method of laying concrete in water that is running rapidly. A concrete dam was built recently across the Niagara river just above the falls. The consulting engineer, knowing that it would be impossible to place concrete in such rapidly flowing water, because the cement would be rapidly washed away, had to try some new device. He built the dam on the shore in a vertical position, and when the cement had set, toppled it over into the river. To lay concrete in running water, the engineer must either place it inside a caisson or else shield it with a temporary dam until set.

Concrete can be laid in still or almost still water with little trouble. The concrete must be lowered to place, not dropped, in a closed bucket, so that the cement will not be washed loose in the process. Special concrete buckets are made for this purpose. They open at the bottom and deliver their contents without turning it over.

Foundations have been laid under water by putting the concrete in bags made of sackcloth. Enough cement will percolate through the bags to make them stick together.

60. Waterproofing Concrete.—Various methods have been proposed to render concrete waterproof. If mixed fairly wet when it is laid, it will be more waterproof than if mixed dry. Sometimes, a coat of pure cement or of one-to-one mortar $\frac{1}{2}$ inch thick is applied after the work has set. Of course, the concrete may be painted with waterproof paint, but this is often unsightly. A material better than paint, although also unsightly, is asphalt. The concrete must be thoroughly dry before the asphalt is applied. The asphalt is applied while boiling hot, and is put on with a brush.

Another method of making concrete waterproof is by using the **Sylvester process.** After the cement and sand and broken stone are mixed, alum is added to the mass in the proportion of 20 pounds of alum to 1 cubic yard of concrete. The water that is added to make the concrete has in it Castile soap in the proportion of $\frac{3}{4}$ pound of soap to 1 gallon of water. The alum and the soap form an insoluble gelatinous substance that fills all the pores of the mass.

It is not often necessary, however, to resort to any waterproofing process for concrete. If the materials are mixed fairly wet and well tamped, they will usually be impervious to water with no additional precautions.

CONCRETE BLOCKS

61. Although artificial stone has been made for centuries, it is only within the last few years that there has been a rapid development in the manufacture of the concrete building block. Most concrete building blocks are made hollow. A very common form of block is shown in Fig. 22. Concrete blocks are made in molding machines—usually in an iron mold. They are made of dry concrete, so that they can be quickly removed from the mold. In this way, the molding machine has a larger capacity than if wet concrete were used. By changing the face of the mold, the blocks can be made to represent different kinds of stonework. The color of the block can also be made to imitate various kinds of stone, by changing the kind of aggregate used. Some makers change the color by using artificial coloring.

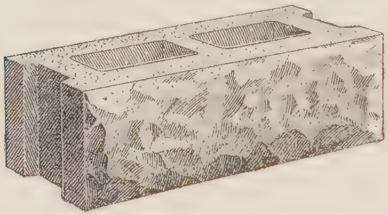


Fig. 22

Almost all building blocks, except for the coloring, are made of ordinary concrete. There are, however, some makers that use special ingredients or special treatment in making their blocks. It is common among some makers to

cast their blocks under heavy pressure. The effect of this is to make the blocks denser, so that they will absorb less water.

The engineer should be very cautious in the use of concrete building blocks. Many of them are poorly and cheaply made, to compete with natural stone. They are sandy, absorb water freely, and present a dull regular appearance, which is often very far from the appearance of natural stone.

STRENGTH OF PLAIN CONCRETE

62. The ultimate strength of concrete for tension, compression, and shear can be taken from Table III. The engineer has, however, to select a proper factor of safety.

As already stated, the strength of concrete increases with age. It is, therefore, necessary for the engineer to know when the concrete will be loaded. It is customary to assume a factor of safety based on the strength of the concrete after 6 months. The engineer must be careful that the concrete, in the first few months after being laid, is not subjected to too great stresses. For general work, a factor of safety of 5 on concrete 6 months old is recommended. This will give the required strength for the first few months, and yet will not be wasteful of material at any time. A factor of safety of 4 on concrete 6 months old is considered a little risky, but may be used for steady loads, such as earth fills, water pressure, etc.

CONCRETE COLUMNS

63. Column Centrally Loaded.—Plain concrete columns may be divided into two classes; namely, those that are centrally loaded, and those that are eccentrically loaded. In either case, the height of the column should never be more than twelve times the least dimension of the cross-section. For a centrally loaded column, the allowable stress per square inch is multiplied by the area of the cross-section of the column to find the allowable load. Thus, if it is decided to allow an intensity of stress of 500 pounds per square inch, and the column is of square section 10 inches on a side, the allowable load will be $10 \times 10 \times 500 = 50{,}000$ pounds. The

breaking load on columns between two and twelve times as high as the least dimension of their cross-section seems to be independent of their height. A column between these two limits, however, cannot withstand as high an intensity of stress as a cube, for it is more likely to break by shearing. For this reason, when employing values taken from Table III for column calculations, a larger factor of safety should be used than with other concrete work. This factor is usually taken as 6.

EXAMPLE.—What is the allowable working load, on a concrete column 10 feet high and 12 inches in diameter, made of 1 : 2 : 4 concrete?

SOLUTION.—The cross-sectional area of the column is $.7854 \times 12^2 = 113.1$ sq. in. From Table III, the ultimate crushing strength of 1 : 2 : 4 concrete 6 mo. old is 2,500 lb. per sq. in. Using a factor of safety of 6, the safe intensity of stress is $\frac{2{,}500}{6}$. Then the safe total load the column can carry is

$$113.1 \times \frac{2{,}500}{6} = 47{,}130 \text{ lb. Ans.}$$

64. Eccentrically Loaded Columns.—When the line of action of a load acting on a column does not coincide with its axis but is parallel to it, the column is said to be eccentrically loaded and the distance of the line of action from the axis of the column is called the **eccentricity** of the load. An eccentric load induces in the column bending stresses in addition to those of direct compression, and for short columns, such as are used in concrete construction, the combined effect of the direct stresses and bending may be obtained by the following formulas.

Let P be the total load on the column, P_e the eccentric part of the load, e the eccentricity of P_e, A the area of the cross-section of the column, I its moment of inertia about the neutral axis, which is perpendicular to the line of eccentricity, and c the distance of the outermost fibers from the neutral axis. Then for the fibers, whose stress due to bending is compressional, the combined stress is

$$S_c = \frac{P}{A} + \frac{P_e ec}{I} \qquad (1)$$

which total should never exceed the allowable unit working stress of concrete in compression; and for the fibers whose stress due to bending is tensional,

$$S_t = \frac{P}{A} - \frac{P_e e c}{I} \qquad (2)$$

It will be seen that when the second term of formula (2) is greater than the first term, S_t becomes tension. Plain concrete should preferably not be employed in tension, and, at any rate, should not be stressed more than from 20 to 40 pounds per square inch.

For a cylindrical column having a diameter d, $c = \frac{d}{2}$ and $I = \frac{Ad^2}{16}$. Substituting these values, formulas (1) and (2) become, respectively,

$$S_c = \frac{P}{A} + \frac{8P_e e}{Ad} \qquad (3)$$

and

$$S_t = \frac{P}{A} - \frac{8P_e e}{Ad} \qquad (4)$$

For a rectangular column in which the plane of eccentricity is midway between and parallel to the sides having the dimension d, $c = \frac{d}{2}$ and $I = \frac{Ad^2}{12}$. These values substituted, formulas (1) and (2) become, respectively,

$$S_c = \frac{P}{A} + \frac{6P_e e}{Ad} \qquad (5)$$

and

$$S_t = \frac{P}{A} - \frac{6P_e e}{Ad} \qquad (6)$$

If the total load is eccentric, the formulas (3) and (4) for cylindrical columns reduce to

$$S_c = \frac{P_e}{A}\left(1 + \frac{8e}{d}\right) \qquad (7)$$

$$S_t = \frac{P_e}{A}\left(1 - \frac{8e}{d}\right) \qquad (8)$$

and formulas (5) and (6) for rectangular columns reduce to,

$$S_c = \frac{P_e}{A}\left(1 + \frac{6e}{d}\right) \qquad (9)$$

$$S_t = \frac{P_e}{A}\left(1 - \frac{6e}{d}\right) \qquad (10)$$

It will be noted in these formulas that, to design a column that will stand a given load, it is first necessary to select, by guesswork, the section of column that seems to be about correct, and then solve the equation of S. This value of S must be less than the allowable working stress it is proposed to use. If it is larger, a larger area of column must be selected, and the problem again worked out; if it is very much smaller, it may be assumed that too large a section for economy has been selected, and in this case also it is well to again go over the work with a smaller section. It should be borne in mind that the height of the column must not be greater than twelve times the least dimension of the cross-section.

EXAMPLE 1.—To design a cylindrical column of 1 : 2 : 4 concrete, 12 feet high, to carry a central load of 100,000 pounds and an eccentric load of 100,000 pounds, the eccentricity being 4 inches.

SOLUTION.—Since the column is 12 ft. high, it must be at least 12 in. in diameter to be less than twelve diameters high. The ultimate crushing strength may be taken at 2,500 lb. per sq. in. in 6 mo. The safe working stress would, therefore, be $2,500 \div 6 = 417$ lb. per sq. in.

A column 28 in. in diameter will be tried first. The area of the cross-section is $.7854 \times 28^2$, or 615.75 sq. in. To apply formula **3**, we have $A = 615.75$, $P = 200,000$, $P_e = 100,000$, $e = 4$, and $d = 28$. Substituting in the formula,

$$S = \frac{200,000}{615.75} + \frac{8 \times 4 \times 100,000}{615.75 \times 28} = 325 + 186 = 511 \text{ lb. per sq. in.}$$

This stress is larger than the allowable stress, which shows that the column section selected is too small. If a section 31 in. in diameter is assumed, we have,

$$A = .7854 \times 31^2 = 754.77 \text{ sq. in.}$$

Substituting in the formula,

$$S = \frac{200,000}{754.77} + \frac{8 \times 4 \times 100,000}{754.77 \times 31} = 401 \text{ lb. per sq. in.}$$

Since this is less than 417 lb., a column of this diameter is safe. Ans.

EXAMPLE 2.—What should be the size of a column of square section, to carry the same load as in example 1, the other conditions being the same?

SOLUTION.—Assuming a column 26 in. square, $A = 26^2 = 676$. Substituting in formula **5**,

$$S = \frac{200,000}{676} + \frac{6 \times 4 \times 100,000}{676 \times 26} = 433 \text{ lb. per sq. in.}$$

This is larger than 417, therefore a larger section, say 27 in. square, will be tried. $A = 27^2 = 729$. Substituting in the formula,

$$S = \frac{200{,}000}{729} + \frac{6 \times 4 \times 100{,}000}{729 \times 27} = 396 \text{ lb. per sq. in.}$$

Therefore, a column 27 in. square is sufficient. Ans.

EXAMPLES FOR PRACTICE

1. A column is made of 1 : 3 : 6 concrete. Its cross-section is a square 8 inches on a side and its height is 8 feet. What central load will it sustain, using a factor of safety of 6? Ans. 23,470 lb.

2. What is the required diameter of a cylindrical column made of 1 : 3 : 6 concrete, the height being 8 feet, if the column is to carry a load of 50,000 pounds, all of which is eccentrically placed, with an eccentricity of 3 inches? Use a factor of safety of 6. Ans. 20 in.

3. A column 15 inches in diameter and 15 feet high is made of 1 : 2 : 4 concrete; it carries a central load of 40,000 pounds and an eccentric load of 30,000 pounds. The eccentricity being 3 inches, what is the intensity of stress produced? Ans. 668 lb. per sq. in.

REINFORCED CONCRETE

INTRODUCTION

1. Definitions.—The terms **ferro-concrete, concrete steel, armored concrete,** and **reinforced concrete** are indifferently applied to concrete in which is embedded steel in the form of rods, bars, shapes, or netting. Although all these terms are employed, the last one—*reinforced concrete*—is superseding all the others. The metal used with the concrete is called the **reinforcement.** The term *reinforced concrete* does not apply to those combinations of steel and concrete in which the steel is designed to support all the loads. In

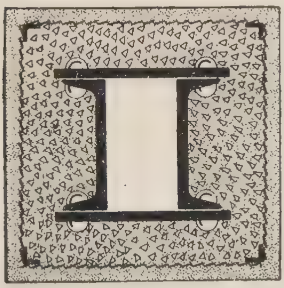


Fig. 1

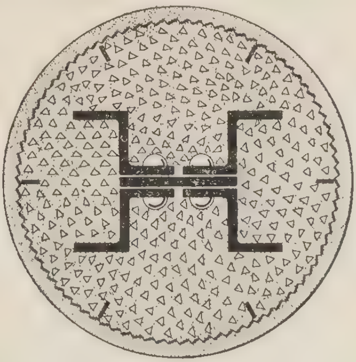


Fig. 2

these combinations (see Figs. 1 and 2), the concrete simply protects the steel from corrosion and fire, and the steel is designed to support the concrete as well as the other loads. In reinforced concrete, on the contrary, the proportions of steel and concrete are so designed that the stresses will be distributed properly between the two materials.

2. Object of Reinforcement.—Concrete structures are used principally on account of their rigidity and durability. The primary object of the reinforcement is to reduce the cost of the structure. Concrete has a low tensile strength, and the amount of it that would be required in structures subjected to tension would be very large. It is, therefore, more economical to use steel to resist tensile stresses. On the other hand, the compressive strength of concrete is comparatively high, and in many cases it is cheaper to use this material to resist compressive stresses. Hence, a very economical structure can be obtained by combining the two materials and so designing the structure that the compressive stresses will be resisted by the concrete, and the tensile stresses by the steel. This is the principle on which reinforced-concrete construction is based. In such construction it is customary to assume that all the tensile stresses are resisted by the steel, and all the compressive stresses by the concrete.

3. Concrete structures that resist compression only, such as columns and piles, are commonly reinforced by steel, as it is found that they can better withstand the effects of shocks than when not reinforced. In some cases, the steel assists in resisting the compression; in others, it simply serves to hold the concrete together in such a way that it can withstand greater stresses.

In addition to the reinforcement that is added to assist in resisting stresses due to applied loads, steel is sometimes placed close to the outside of concrete structures to prevent surface cracks due to the drying of the concrete and to changes in temperature.

4. Uses of Reinforced Concrete.—Reinforced concrete is extensively used in engineering construction. Some of the uses to which it has been put and to which it is adapted are as follows: floors, beams, columns, and column footings in bridge building, and dock construction; piles and sheet piling; retaining walls, bridge abutments, piers, and trestle bents; culverts, sewers, and other conduits; floors, roofs, and walls in subway construction; tanks, standpipes, and roofs of reservoirs; arches;

and dams. In some of these constructions, the amount of steel that is embedded in the concrete is decided according to practical considerations; in others, it is computed by means of formulas, so that the stresses will be properly distributed between the concrete and the steel.

5. Advantages of Reinforced Concrete.—Reinforced concrete possesses all the advantages of plain concrete. In addition, it has the advantage that it requires much less material, for the same strength, than plain concrete, and that it is freer from cracks and the consequent deterioration caused by moisture and frost entering them. Reinforced concrete can be put in place by unskilled labor, but a competent engineer or superintendent of construction is required to direct and oversee the work. It is his duty to see that the steel is placed properly according to the plans, and that the concrete is properly mixed, deposited, and tamped.

6. Adhesion of Concrete and Steel.—In order that the concrete may transmit the stresses to the steel reinforcement, it is necessary that the steel and the concrete be in perfect contact and that there be some kind of connection between them. This connection is provided in some cases by projections on the sides of the reinforcing rods or shapes. These projections bear against the concrete and transmit the stresses to the steel. In by far the greater number of cases, however, the adhesion between the concrete and the surface of the steel is relied on to transmit the stresses to the reinforcement. The *ultimate* adhesion between smooth steel and concrete usually varies between 250 and 450 pounds per square inch of surface of steel in contact with the concrete. The *working* adhesion is usually taken between 50 and 75 pounds per square inch; 60 pounds per square inch is a safe average working value.

7. Extension of Concrete and Steel Due to Stress. When concrete is reinforced with steel, it extends from ten to twenty times as much as plain concrete before showing signs of cracking. It does not follow, however, that concrete, when reinforced, can resist ten or twenty times as much tension as

plain concrete; for, as a matter of fact, concrete thus stretched cannot resist tension, the stretching being really due to cracks that are too small to be readily detected. Plain concrete, when subjected to tension, usually breaks at one section, and the break is plainly visible. In the case of reinforced concrete, the adhesion between the concrete and the steel distributes the elongation throughout the length of the reinforcement, with the result that the concrete breaks simultaneously at a great number of sections, each opening or crack being so small as to be invisible to the naked eye. While extended in this way, the concrete is held in place by the steel, and is no more capable of resisting tension than if there were one large crack plainly visible. When the tensile stress in the steel decreases, however, the steel contracts and, provided it has not been loaded beyond the elastic limit, resumes its original length. The small cracks in the concrete close up, and the concrete is then as capable as before the stretching of withstanding compression, although it can stand no tension. This fine cracking presumably does not alter the strength of the structure, if the latter has been designed on the assumption that the steel resists all the tension.

8. Temperature Expansion and Contraction of Concrete and Steel.—The stresses caused in the concrete and in the steel reinforcement by expansion and contraction due to ordinary changes in temperature are usually so slight that they may be neglected. This is due to the fact that the coefficients of expansion of concrete and steel are very nearly equal, and, since the two materials are at the same temperature, they expand and contract by the same amount. From observations on reinforced concrete that was subjected to fire at San Francisco, however, it is evident that, when the combination becomes highly heated, the steel expands so much more than the concrete, and the latter deteriorates to such an extent, that the two materials separate, and the concrete falls off in large pieces, leaving the steel exposed.

9. Protection of Steel from Corrosion.—There is considerable difference of opinion on the subject of the pro-

tection of steel embedded in concrete. There are many cases on record in which steel has been found in perfect condition, almost free from corrosion, on being removed from concrete in which it had been embedded for many years. There have been cases in which the steel was found rusted all through. All the evidence available at the present time, however, seems to indicate that, if the concrete is properly made and the steel is a sufficient distance from the surface, the danger of corrosion is slight. Steel embedded in cinder concrete is more likely to rust than steel embedded in stone concrete. This is due to the fact that cinder concrete is more porous than stone concrete, and allows air and moisture to reach the steel. In addition, there are likely to be some ingredients in the cinders that will combine with the moisture in the air, forming acids that corrode the steel.

10. Protection of Reinforced Concrete from Fire. Previous to 1907, it was the general belief that reinforced concrete was fireproof. Later investigations, however, especially those made at San Francisco after the great fire of 1906, show that this is not true. As stated in Art. **8**, a very high temperature weakens the concrete, and also causes the steel to expand so much more than the concrete that the latter breaks and drops off. For this reason it is advisable, when reinforced concrete is used where there is a possibility of fire, to take extraordinary precautions. A better grade of concrete should be used, greater care should be taken in mixing and depositing it, and there should be a greater distance—at least 2 inches —from the steel to the surface of the concrete.

11. Grades of Concrete; Mixing.—The best mixture for reinforced-concrete work consists of 1 part Portland cement, 2 parts clean, sharp sand, and 4 parts broken stone not over $\frac{3}{4}$ inch in any direction. For large, solid structures, such as retaining walls, foundations, abutments, etc., $1:2\frac{1}{2}:5$ and $1:3:6$ mixtures are sometimes used, and in these mixtures larger stones are allowed. It has been found by experiment and in practice that when well-made concrete is broken, the

break often passes through the broken stone, showing that the adhesion of the cement and stone, and the strength of the cement between the stones, is greater than the strength of the stones. To develop the full strength of the concrete, then, it is necessary to use a good grade of very hard, tough stone. Greater care should be taken than with plain concrete to see that the sand and stone are perfectly clean and free from clay or loam, as these impurities prevent a good bond between the cement and the stone. Cinder concrete is not desirable for reinforced-concrete work, unless it is absolutely necessary to reduce the weight, and unless strength is a secondary consideration. Broken brick is frequently used, and is a good material, but good, hard, broken stone, such as granite or trap, is preferable, if available.

Even with the best materials, the full strength of reinforced concrete is available only when the materials are properly and thoroughly mixed and carefully placed in the forms, so as to eliminate the possibility of open spaces or voids in the finished concrete. Better results are obtained when the concrete is deposited quite wet than when a comparatively dry mixture is used; the wet concrete makes a more compact and homogeneous mass and adheres to the reinforcement better than the drier mixture. Too much water, however, tends to cause the materials to deposit in layers, and sometimes gives a very porous concrete, on account of spaces being filled with water, which subsequently evaporates. The best results are obtained when the concrete has such a consistency that, when placed in the forms, a very small amount of free water rises to the surface. Whether a wet or a dry mixture is used, the concrete should be thoroughly agitated, or "worked," after it is placed in the forms. A comparatively dry mixture requires more working than a wet mixture, and, in order that the concrete may be compact and homogeneous, must be thoroughly worked and tamped.

12. Location of Reinforcement in Cross-Section. It is very desirable to place the reinforcing steel at the sections where the tension is greatest. In beams and girders, this brings the steel close to the surface of the tension side of

the beam. In columns there is usually no tension, but, on account of the liability of the columns to deflect, thus bringing one side into tension, it is customary to place the steel near the surface, or as far as practicable from the axis or center line of the column.

CONSTRUCTION WORK

TYPES OF REINFORCEMENT

13. Plain Rods and Patented Bars.—There are various ways of reinforcing concrete with steel, the principal difference being in the form or shape in which the reinforcement is manufactured. The form of reinforcement most frequently used consists of ordinary round steel rods. The action of these rods depends on the friction or adhesion between the steel and the concrete. When the concrete is well made, this adhesion is sufficient for many purposes. Many engineers, however, prefer to use rods or bars that are especially designed for reinforcing purposes, and the action of which is not limited by adhesion. The makers of these patented bars claim that the tendency of the smooth rods to slide or slip through the concrete renders them unsuitable for the purpose of reinforcing. At the present time, the choice between smooth rods and patented bars is to a great extent a matter of individual preference, although it is obvious that the patented bars answer the purpose somewhat better than plain rods. They are, however, more expensive, and whether their higher efficiency is enough to compensate for their greater cost is still an open question. There are now a great many kinds of patented bars on the market, and their number is continually increasing. In this Section, only a few of those which are most commonly used will be described.

14. Ransome, or Twisted, Bars.—Fig. 3 shows a **Ransome,** or **twisted, bar.** This bar is first rolled straight and with a square cross-section. It is afterwards twisted, while cold, into the desired form by special machinery. The

twisting process increases the elastic limit, so that a higher working stress can be used. Since the bar has no long, smooth,

FIG. 3

straight surface on its side, it has little tendency to slide through the concrete when subjected to stress.

15. Corrugated Bars.—Figs. 4 and 5 show two forms of patented bars, called **corrugated bars** on account of the many projections and indentations of their sides. The bar shown

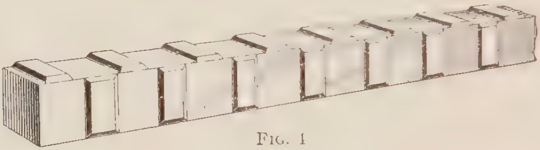


FIG. 1

in Fig. 4, called the **Johnson bar,** is approximately square in cross-section, and has projections on all sides; that shown in Fig. 5, called the **universal bar,** has rectangular indentations on two opposite sides, the main body of the bar being oblong in cross-section, with the top and bottom rounded off.

FIG. 5

Corrugated bars do not work wholly by adhesion, but mainly by the mutual pressure between the concrete and the steel at the surfaces of the corrugations. The patent under which these bars are manufactured includes all systems that work by the mutual pressure between the concrete and the steel.

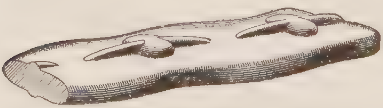


FIG. 6

16. Thacher Bars. Fig. 6 shows a **Thacher bar,** which is somewhat similar to that shown in Fig. 5; it is nearly elliptical in cross-section, and has projections on two opposite sides; the projections occur

at larger intervals than in corrugated bars. These projections assist in transmitting stress and in preventing slipping; but adhesion, too, is depended on to a certain extent. The Thacher bar is very much used.

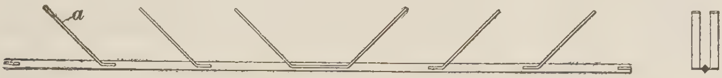


FIG. 7

17. Kahn Bars.—Fig. 7 shows a **Kahn bar** as it is placed in the concrete. Fig. 8 shows an enlarged cross-section of the bar before the projections are bent up. This bar consists of a single piece of steel rolled to the cross-section shown in Fig. 8, both the square portion *c* at the center and the thin edge pieces *d* being continued for the full length of the bar. Any desired length can be cut off, the edges sheared at several places, and these sheared pieces bent up in the manner shown at *a* in Fig. 7. These bars are used extensively in beams, columns, and floor construction, and give good results. They are far superior to the plain or straight bars, on account of the bent-up projections, or **fins**, as they may be called. These projections serve a double purpose in preventing any cracks in the portion of the concrete through which they pass, and in transmitting stress to the body of the bar.

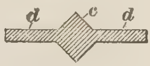


FIG. 8

18. Cummings Bars.—In Fig. 9 is shown a set of three **Cummings bars.** Each bar is a single smooth round rod bent into the shape of a long rectangle. These bars are used in sets, as shown, and the ends are bent up so they will pass

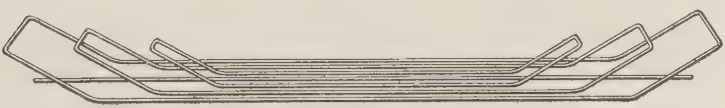


FIG. 9

diagonally across a beam and help in resisting diagonal stresses. When more than one bar is used in a beam, the widths of the rectangles are made unequal, so the bars will not be in contact, as in this way the concrete encloses each rod separately

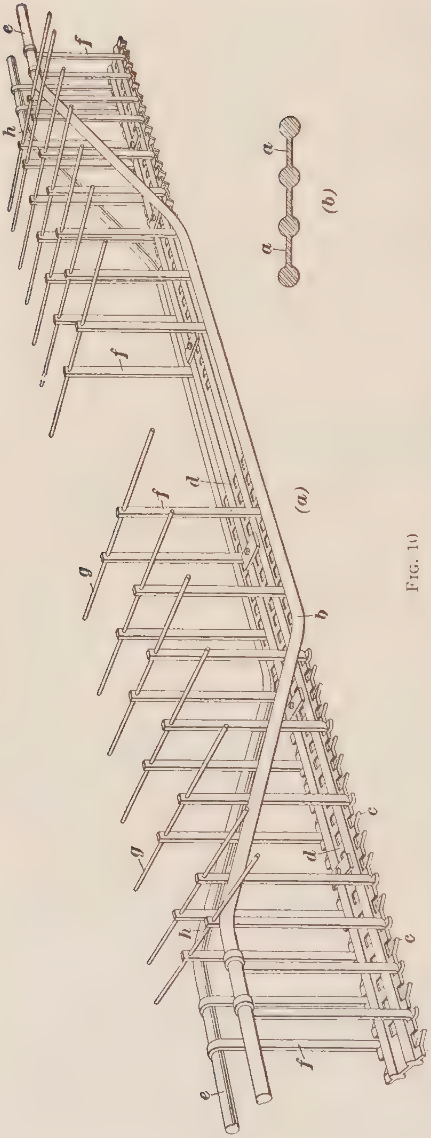


FIG. 10

and has therefore a good bond. When these bars are employed, one straight bar is usually inserted at the center of the beam near the bottom, and continued the entire length of the girder or beam without being bent up.

19. Unit Concrete Steel Reinforcement. There are several systems of reinforcing girders that do not consist of single rods or bars, but of a number of different shapes fastened together to form a frame or skeleton. This frame is surrounded and filled with concrete, and serves the same purpose as a number of separate pieces. Fig. 10 shows the **unit girder frame,** which may be taken as typical of this form of reinforcement. The main part of the frame consists of one piece *d* rolled to the shape shown in cross-section in Fig. 10 (*b*); that is, four round rods connected by three thin webs *a*. About one-third of each of the two outside rods is sheared from the web, bent up at *b*, and continued

diagonally through the beam to *h*, where it is bent again and then continued horizontally near the top of the concrete, as shown at *e*. For almost the entire length of the girder, the thin webs are punched at short intervals so that the concrete may pass through and get a better hold on the steel. The outside portion of the web at the ends, where the outer rods are turned up, is punched at intervals and bent down, as shown at *c*. To complete the frame, a number of **U**-shaped **stirrups** *f* are added. These stirrups pass under the main portion, and come up through the punched holes in the web. Those near the end are bent around the upper part *e* of the main rods; the others have holes in the top through which the rods *g* that reinforce the floor slab are passed.

20. Expanded metal is a form of reinforcement that is manufactured from a plain sheet of steel. Fig. 11 shows a

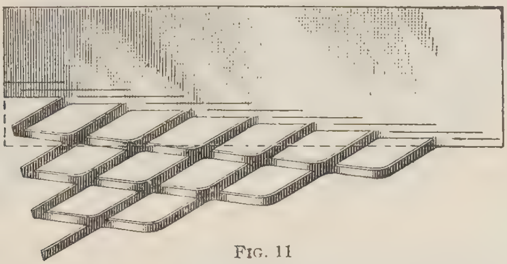


FIG. 11

sheet of steel partly expanded; the upper portion shows the unexpanded part of the plate, the lower portion shows the expanded part, and the horizontal lines in the plate just above the expanded part show where the sheet will next be opened out. Expanded metal resembles poultry netting, but is usually much heavier. It can be made from thick steel plates, and with either small or large openings. It is one of the best reinforcing materials for floor or other slabs subjected to bending stresses.

21. Wire cloth consists of two systems of straight wires crossing each other at frequent intervals. The wires are sometimes welded together at their intersections; sometimes a short piece of wire is twisted around each intersection; and sometimes the wires of one system are twisted around those of the other.

METHODS OF CONSTRUCTION

22. Floor Slabs.—Fig. 12 shows the customary method of arranging the concrete and steel reinforcement in a reinforced-concrete floor slab. The reinforcement shown in the figure is expanded metal, but the arrangement is the same whether this material, wire cloth, or rods are used. The wooden forms are first built up with their top surfaces at the desired elevation of the bottom of the concrete. The reinforcement is then put on top of them, and supported so that it is about 1 inch above the forms at the center of the span, and about 1 inch below the top of the concrete at the beams. When expanded metal or wire cloth is used, the required bend to accomplish the desired end can be readily obtained. When rods are used, it is necessary to bend them by means of a special

FIG. 12

apparatus arranged for the purpose. In Fig. 12 the slab is shown supported by steel beams, but the principle is the same when reinforced-concrete beams are used.

23. In many cases, when steel beams are used, the slab is so placed that its top is level with the top of the beams, as shown by the line *e e* in Fig. 13. The slab is then supported by concrete haunches *a* that extend down to and rest on top of the bottom flanges of the **I** beams. The concrete is frequently plastered on the bottom for the sake of appearance. Reinforced-concrete floor slabs are frequently finished smooth on top with cement, and this surface is used as a floor. In many cases, however, wooden floors are used; these are laid as shown in Fig. 13. The strips *b*, commonly called **sleepers,** are laid flat on top of the floor slab, and the spaces between them are

Fig. 13

filled with concrete. The wooden floor c is then nailed directly to the top of the sleepers. When the arrangement shown in Fig. 13 is used, the reinforcement is laid close to the bottom of the slab for its full width.

24. Fig. 14 shows a patented style of floor slab that is intended to do away with the forms used in construction. These slabs can be manufactured in a yard where there is plenty of room, hauled to the structure, and placed in position without the aid of any forms, except a few at the yard. The reinforcement shown in Fig. 14 consists of straight rods; any other reinforcement might be used. The steel is allowed to project from the slab at b, and the portion of concrete at the lower part of the beam is put in after the slabs are in place.

25. Beams.—The reinforcement in beams is usually placed near the bottom of the beam wherever there is positive bending moment, near the top of the beam wherever there is negative bending moment, and diagonally or vertically through the beam wherever there are shearing stresses. The horizontal reinforcement is usually placed from 1 to 3 inches from the surface of the concrete, and the required amount of metal is sometimes made up of a few rods of large cross-section, and sometimes of a larger number of bars of smaller cross-section. The latter method is the better, because the stresses are more evenly distributed through the beam when the steel is scattered. There should always be at least 2 inches between consecutive rods, however, in order to allow the concrete to enter between them.

Fig. 15 shows an outline of a simple beam, having the horizontal reinforcement a near the

bottom of the beam and the inclined reinforcement b to resist the shearing stresses. The inclined reinforcement is sometimes composed of separate rods; sometimes, the horizontal rods are bent up to form the inclined rods. The latter method is to be

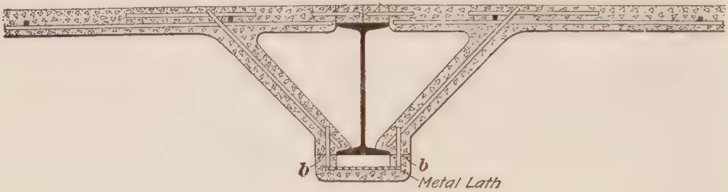


FIG. 14

preferred, as it is better to have the shear rods continuous with the others. Since the shearing stress is greatest at the end of the beam, the diagonals are placed closer together near the end of the beam than near the center.

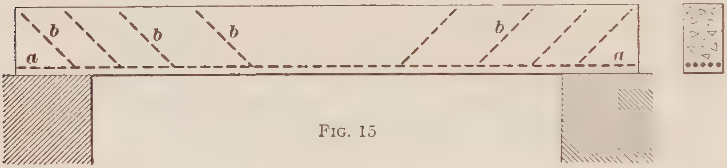


FIG. 15

26. When the ends of the beam are fixed, as when they are built into walls at the ends or are continuous over two or more supports, some reinforcement is placed near the top of the section. In some cases, the entire reinforcement near

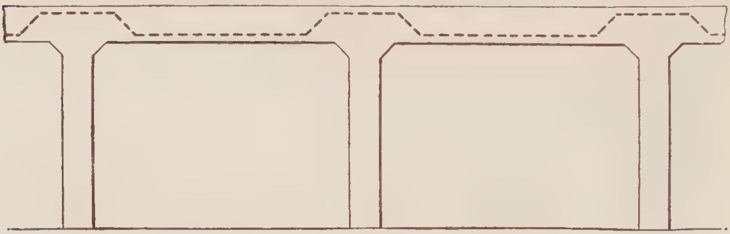


FIG. 16

the bottom is bent up and carried diagonally across the beam to the top, as shown in Fig. 16 in dotted lines. As a rule, however, it is preferable to use separate rods for the upper

reinforcement, as shown in Fig. 17. In many cases, the inclined reinforcement is left out, and the rods at the bottom

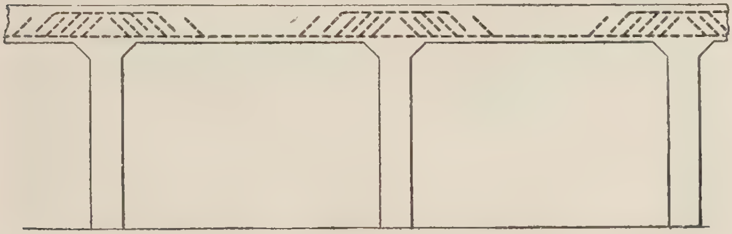


FIG. 17

are not continued the full length of the beam. This is bad practice, and should be avoided. The upper rods should be bent down, and the lower rods bent up, the two sets alternating with each other. The upper rods should continue for about one-quarter of the span at each end. Fig. 18 is a view of a beam with Kahn bars at both top and bottom, and with the shear bars or fins alternating.

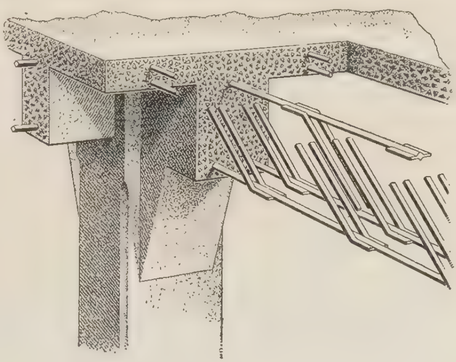


FIG. 18

27. Although there is a great difference of opinion among engineers as to whether inclined or vertical shear bars are more efficient, there can be no question as to the necessity of making some provision for shear, since the shearing strength of concrete is but little greater than the tensile strength. When full-sized rein-

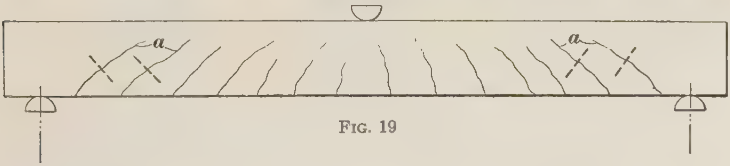


FIG. 19

forced-concrete beams that are not provided with diagonal or vertical reinforcement are broken in a testing machine, there are found a large number of cracks; it is these cracks that finally cause the beam to break. They start at the bottom of the beam, and continue upwards, so that when the beam fails they have an appearance like the lines *a* in Fig. 19, being inclined downwards toward the ends of the span. To prevent these cracks, it is necessary to place rods in the concrete crossing them at right angles, as shown by the dotted lines in Fig. 19. This brings the rods at an inclination of approximately 45° to the vertical. The cracks can also be prevented by means of vertical reinforcement. If inclined reinforcement is used it should be securely fastened to the horizontal rods to prevent slipping.

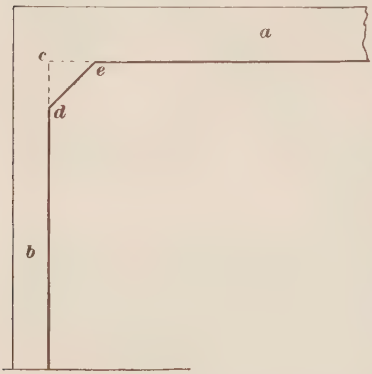


FIG. 20

28. Supports for Beams.—Care should be taken that the ends of beams at the supports are properly proportioned, so that the reactions that are concentrated there will not cause the ends to fail by crushing. The reaction at each support should be found and divided by the safe crushing strength of the concrete. The result is the area of bearing that should be provided. When the beam is built into a vertical wall, as shown in Fig. 20, the increased bearing may be obtained by corbeling out. In the figure, *a* is the beam, *b* the wall, *c* the point where the corner would be if the wall were not corbeled, and *d e* the corbel. In this form of construction, the length of the corbel should be two or three times the width of the beam.

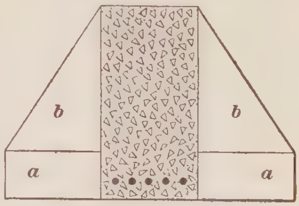


FIG. 21

The bearing area may also be increased in the manner illustrated in Fig. 21, by using a flange or shoe *a* and vertical webs or diaphragms *b*. This bearing is advantageous only

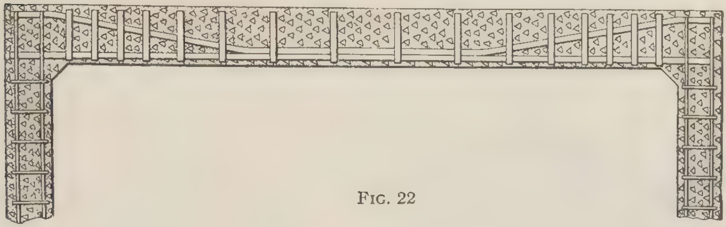


Fig. 22

for very large beams or girders, and then the flanges *a* and the webs *b* should be not less than about 6 inches in thickness.

Where a reinforced-concrete beam rests on a column, it is advisable to continue the reinforcing rods of the beam well into the column, and to continue those of the column through the beam and anchor them both securely as illustrated in Figs. 22 and 23. In Fig. 22, the beam is supported at each column; in Fig. 23, the beam is continuous. In the latter case, the rods *c* that are bent up to the top of the beam are often carried clear through the column into the next span, and the ends of these rods are sometimes split and opened, as shown,

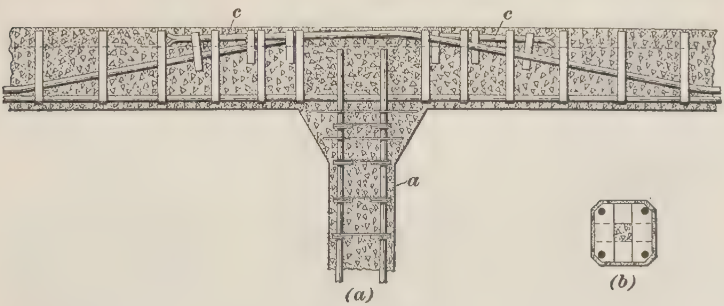


Fig. 23

in order to afford the concrete a better grip. In both cases, it is customary to bevel or haunch the concrete at the bottom of the beam where it connects to the column. This affords greater stiffness to the connection.

29. Columns.—There are many methods of reinforcing columns. The steel is sometimes concentrated at the center of the concrete in the form of structural shapes, as shown in Fig. 24, but this is not considered an economical nor otherwise desirable method. As a matter of fact, when this method is used, the steel is frequently designed to support the entire load, and the concrete serves only to protect the metal from fire and to prevent it from buckling sidewise.

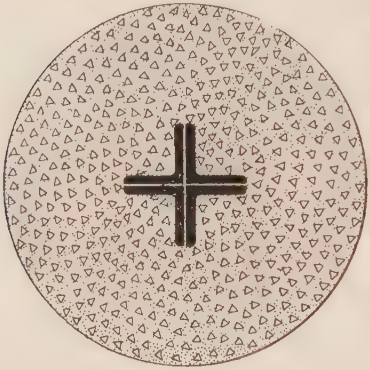
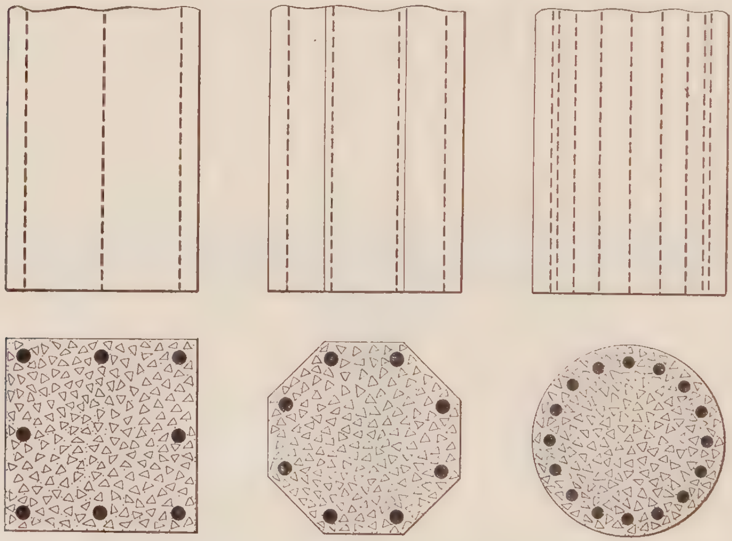


Fig. 24

The usual method of reinforcing columns consists in placing rods vertically in the concrete close to the outside surface, as



(a) (b) (c)

Fig. 25

represented in Fig. 25, in which (a) represents a square column with eight rods, (b) an octagonal column with eight rods,

and (*c*) a round column with sixteen rods. When this type of reinforcement is used, the size and number of the rods can be varied at will. Some engineers simply place the vertical rods in position and then deposit the concrete in the forms. This method of procedure is open to the objection that the tendency of the rods to buckle or deflect sidewise under compression is likely to cause the concrete outside of them to break off, thus allowing the rods to buckle and the column to fail. It is generally recognized at present that it is necessary to provide some means to hold the rods in line and prevent them from buckling.

30. When Kahn bars are used in columns, the fins or projections extend into the heart of the concrete and perform the duty of keeping the bars in line in a more or less satisfactory manner. A perspective view of a column reinforced with four Kahn bars is shown in Fig. 26. When either plain rods or Kahn bars are used, however, the compressive strength of the concrete is not increased, although the load that the column can carry is somewhat greater than if there were no bars, since the steel will carry a certain proportion of the load.

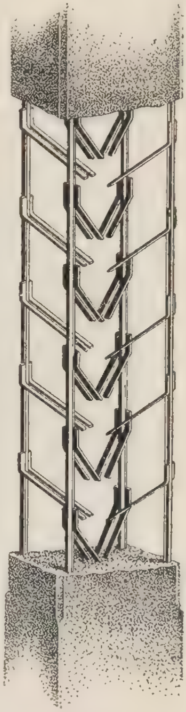


FIG. 26

If a type of reinforcement that will hold the concrete in place is used, the compressive strength will increase and the column will be able to carry a greater load even though there may be no vertical reinforcement. The type of reinforcement that accomplishes this object is called **hoop reinforcement,** and consists either of circular hoops having a slightly smaller diameter than the column, and placed horizontally in the concrete at frequent intervals throughout the height, or of a spiral wire or steel rod wound around the column a small distance from the surface. These two forms are shown in Figs. 27 and 28,

respectively. This reinforcement does not carry any vertical load, but simply makes the concrete capable of withstanding greater compression. The true reason for this is not known, but it is generally assumed that the concrete is held in place by the reinforcement, somewhat like confined sand, and is prevented from failing until the stress on it is considerably higher than the ordinary ultimate strength of plain concrete. It has been concluded from experiments that a given amount of steel is much more efficient when placed as shown in Figs. 27 and 28 than when placed as shown in Fig. 25.

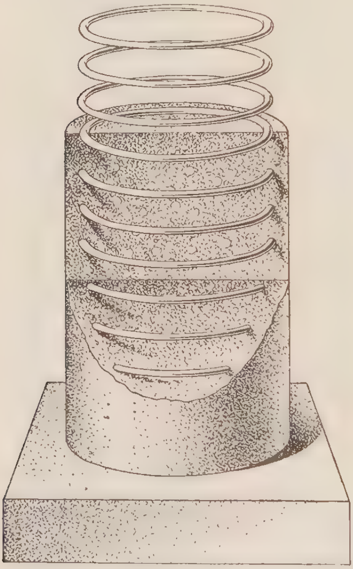


FIG. 27

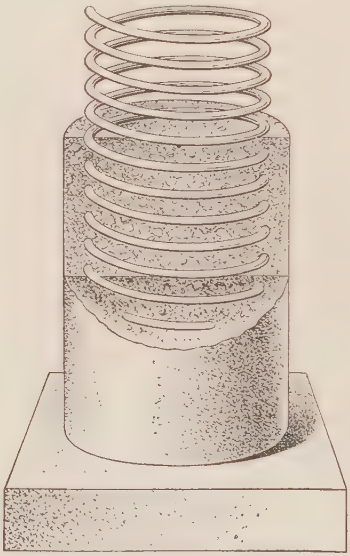


FIG. 28

31. Owing to practical difficulties in construction, neither of the above-described methods give entire satisfaction. The most satisfactory system of column reinforcement, and the method most frequently used in practice at the present time, consists of a combination of the vertical and hoop systems.

Fig. 29 shows a portion of a square column with eight vertical reinforcing rods. At regular intervals throughout the height of the column, the rods are connected by wires running around

the separate rods and around and through the column. In many cases, flat steel or round rods larger than wire are used. The method shown in Fig. 29 can be used with patented as well as with plain bars.

32. When smooth, round rods are used, the method shown in Fig. 30 is frequently employed. Flat bars *p* have round holes punched in the ends and are slipped down over the rods *a*. Four flat bars are placed in this way at frequent intervals throughout the height of the column. The distance between

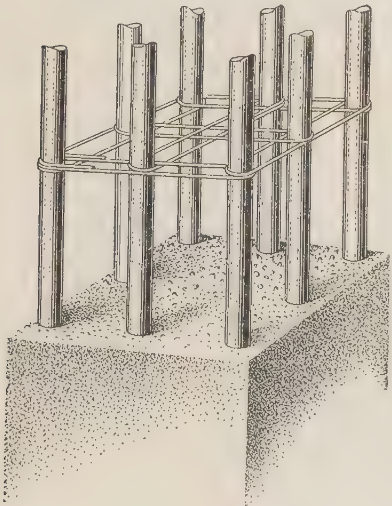


Fig. 29

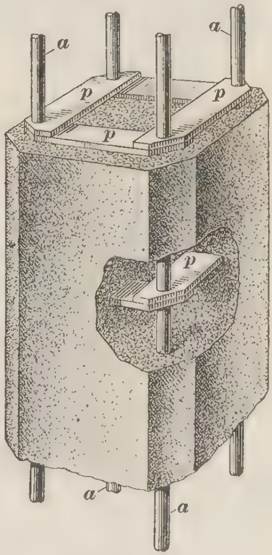


Fig. 30

two consecutive sets of bars should be not greater than the length of one bar. If the bars are placed too far apart, the rods may buckle and cause the column to fail.

This arrangement is not so satisfactory as that illustrated in Fig. 29, for it is likely to create planes of rupture in the concrete where the flat bars are located, on account of the large area of steel compared with the area of the column.

33. Fig. 31 shows a circular column with eight vertical rods wrapped with two layers of expanded metal. This is a

very convenient and practical method of reinforcing. The same method is frequently used with wire cloth in place of expanded metal.

34. Fig. 32 shows the most popular type of column reinforcement in America at the present time; it can be used for square, octagonal, or circular columns. The reinforcement consists of any desired number of rods or patented bars *a* placed vertically in the concrete, and spaced at equal distances apart in a cylindrical form. Before the forms are built up

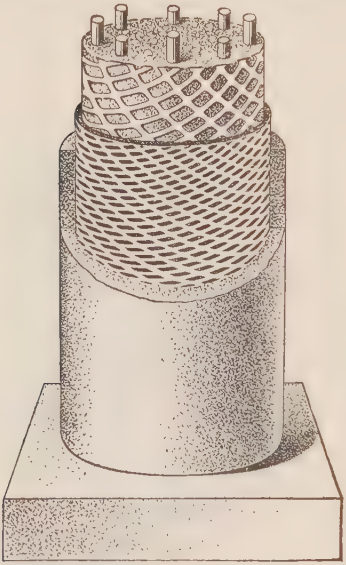


FIG. 31

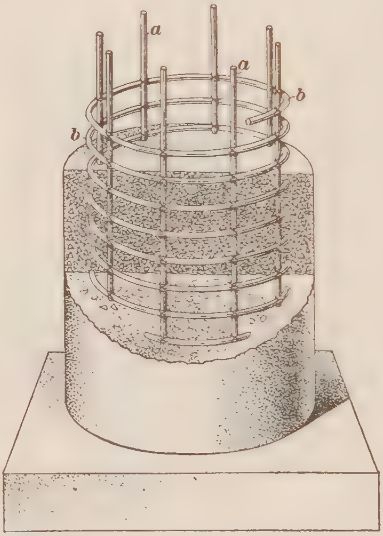


FIG. 32

around these rods, very heavy wire and in some cases $\frac{1}{4}$- to $\frac{1}{2}$-inch round rods *b* are wound spirally around the outside of the vertical rods. The spiral wire or rod is attached to the vertical rods at occasional intersections by means of small wire wound around them. This is simply for the purpose of holding the spiral and vertical rods in their proper positions while the concrete is being deposited.

35. Footings.—Fig. 33 is a cross-section of a reinforced-concrete footing frequently employed under columns. Under

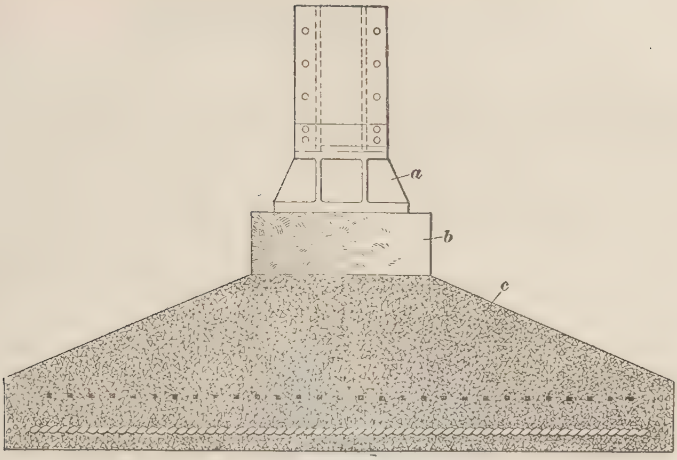
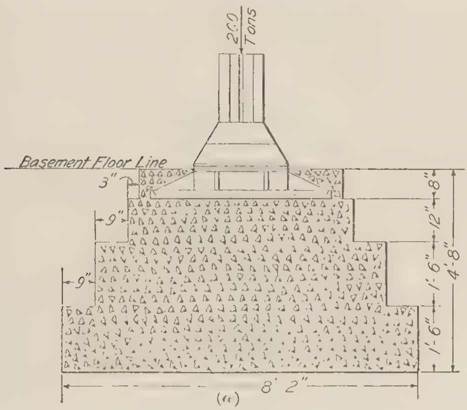
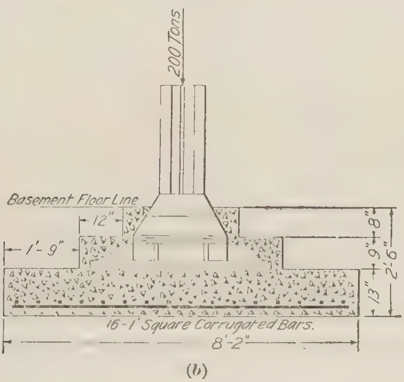


FIG. 33



(a)



(b)

FIG. 34

the pedestal *a* on which the column rests is sometimes placed a block of stone *b*, which rests on the square top of the concrete footing *c*. The bottom of the footing should have the area required for bearing. Near the bottom of the concrete, reinforcing rods are placed in two directions at right angles to each other, as shown in the figure. The required area of steel in the footing and the depth of the concrete can be found in the same manner as for a floor slab; that is, by considering a strip 12 inches wide, and assuming that the load is uniformly distributed over the bearing area.

Fig. 34 shows the difference between a reinforced-concrete footing (*b*) and a plain concrete footing (*a*) for the same load. The saving in excavation, concrete, and labor can be seen at once by comparing the two forms. The form shown in (*b*) is slightly different from that shown in Fig. 33, but the former is frequently used in practice.

36. Piles.—Reinforced concrete has been used to some extent for both single and sheet piles. The details of construction for piles are, in a general way, similar to those for columns. Concrete piles possess the advantage that they do not rot like wooden piles, and are not affected either by fire or by wood-boring insects. Fig. 35 shows the top and bottom of one of the several forms used for reinforced-concrete piles. The lower end has a pointed shoe *a*, the side plates of which are turned in, as at *b*, that they may get a good grip on the concrete. The reinforcement shown in the figure consists of four rods *d* connected at intervals by wire, but any of the forms of column reinforcement described in the preceding articles may be used. In order to avoid breaking the top of the pile while it is being driven, the head is protected by a steel cap *c* filled with sand to form a cushion that distributes the pressure of the blow from the hammer. The diameter of the head is slightly less than that of the body of the pile. The reinforcing rods *d* are often continued beyond the top of the pile, in order to afford a means of connection with other parts.

37. There are several styles of piles on the market that are not driven, but are dropped into a hole in the ground.

One of the best of these is the **Simplex pile.** A long tube is first driven in the soil, the length and diameter being that of the desired pile. This tube is fitted with a concrete tip similar to that shown in Fig. 35. When the tube is driven to the desired depth, the reinforcement is lowered inside it and the tube is gradually withdrawn, leaving the tip at the bottom of the hole and the reinforcement in place. As the tube is slowly

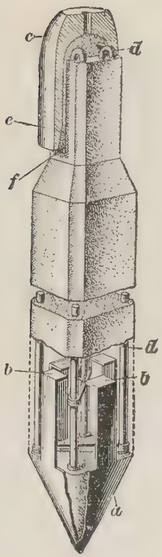


FIG. 35

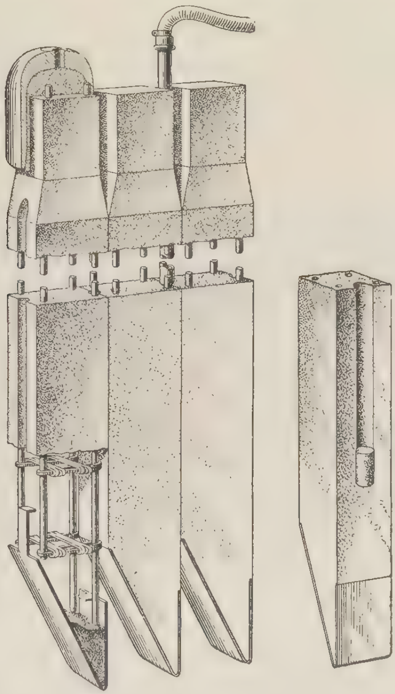


FIG. 36

drawn up, concrete is lowered inside it and deposited in the hole left by the ascending tube.

38. Sheet Piles.—Some reinforced-concrete sheet piles are shown in Fig. 36. In detail of construction and method of manufacture there is very little difference between these piles and those described in Art. **36.** Sheet piles are usually rectangular in cross-section, and pointed by beveling only one

side in order to keep them together while they are being driven. Each pile has a semicylindrical groove on each side where it will be in contact with the adjacent piles. After a pile is driven, a water pipe is driven alongside it in the groove, to serve as a guide for the next pile. When the next pile is driven, the pipe is withdrawn, and, if the sheet piles are to be permanent, the grooves are filled with cement and sand.

39. Dock Construction.—Fig. 37 shows a form of dock construction now being used in the United States. The floor consists of a reinforced-concrete slab supported by steel **I** beams

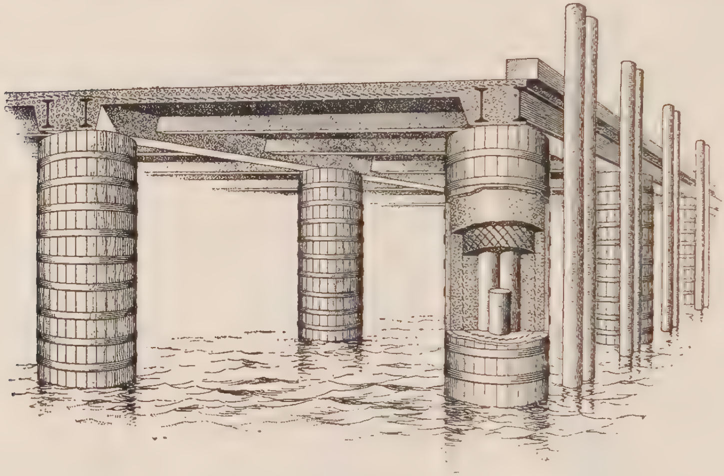


Fig. 37

that are entirely surrounded by concrete. These beams rest on piles made partly of wood and partly of reinforced concrete. From one to three piles are first driven; if there are more than one they should be driven close together. The outer casing of wood with steel bands is then placed around the cluster of piles, and the expanded metal or other fabric for reinforcement is placed in proper position inside of the casing. The casing is then filled with concrete. This form of construction is more expensive than the ordinary types, but it is proof against fire, rotting, and the attacks of wood-boring insects. The completed structure is very rigid, and is practically permanent.

To protect the edge of the dock and the sides of ships that put up to it, wooden piles are driven as shown at the right-hand side.

40. Quay Wall.—Fig. 38 shows a cross-section of a quay wall built at Boston by the United States Government. The

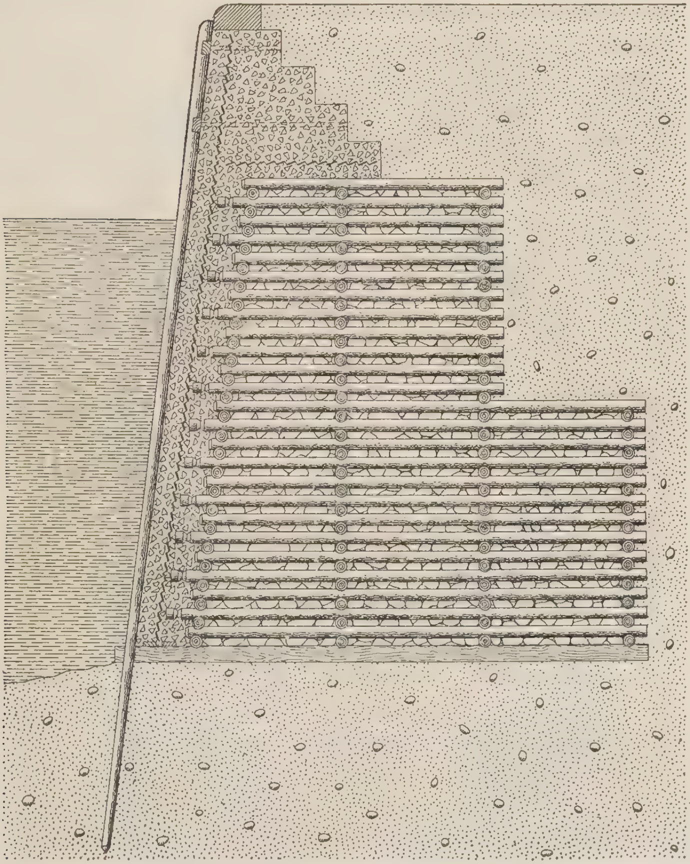


Fig. 38

site for the wall was first excavated, and a crib of timbers put in the excavation, the spaces between the timbers being filled with concrete. The face and top of the wall were finished with concrete reinforced with very heavy sheets of expanded metal.

In order to protect the face of the wall, and also the sides of vessels that might come in contact with it, wooden piles were driven along the face.

41. Sewers and Other Conduits.—Sewers and other conduits are often built of reinforced concrete. There are many forms of construction used for this purpose, a few of which are illustrated in Figs. 39 to 43. The reinforcement usually consists of expanded metal or other fabric, although rods and bars are sometimes used. On account of the irregularly shaped cross-sections of most conduits, expanded metal

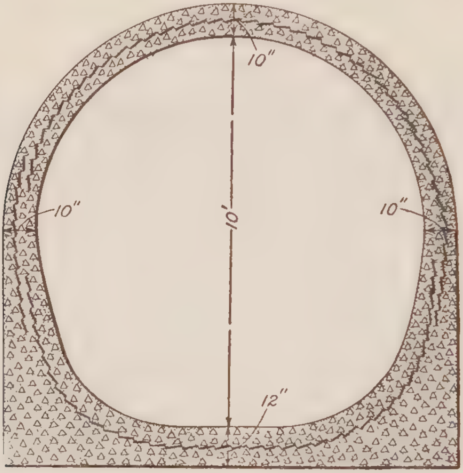


FIG. 39

is best adapted for this purpose. Fig. 39 is the cross-section of a conduit built in New Jersey to withstand considerable internal pressure; it has been found very satisfactory. The upper part of the conduit is 10 inches thick; the lower part is finished flat at the base in order to get a better bearing on the soil. The entire circumference is reinforced with expanded metal.

Fig. 40 is the cross-section of a sewer built to carry drainage in Harrisburg, Pennsylvania. The concrete is 5 inches thick at the top and at the bottom, and 6 inches thick at the haunches. Expanded metal was inserted through the entire circumference.

Fig. 41 is the cross-section of an oval brick sewer built at Boston, Massachusetts. The trench was filled at the bottom

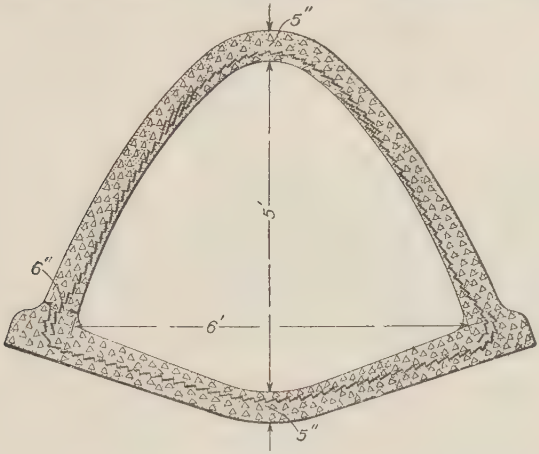


FIG. 40

with concrete, in order to give the sewer a good even bearing. The location of the sewer is such that it is subject to internal

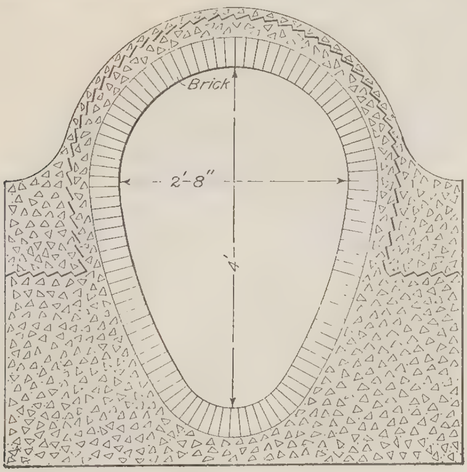


FIG. 41

pressure. Since the brick did not possess sufficient tensile strength, and the earth fill on top was not great, the ring of

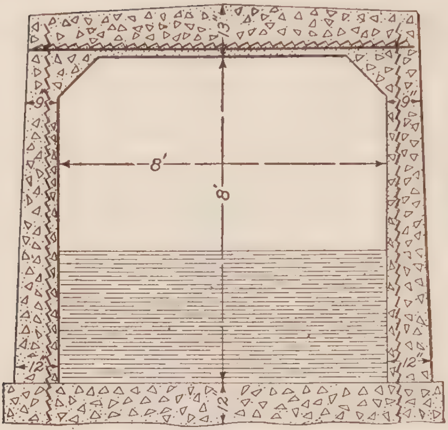


Fig. 42

brick was covered with a layer of concrete 4 inches thick reinforced with expanded metal. The expanded metal was well anchored in the mass of concrete at the sides of the sewer.

Fig. 42 is the cross-section of a sewer built at Buffalo, New York. This is a very convenient form from the standpoint of ease of construction, but it requires more concrete and reinforcement than a sewer with a curved out-

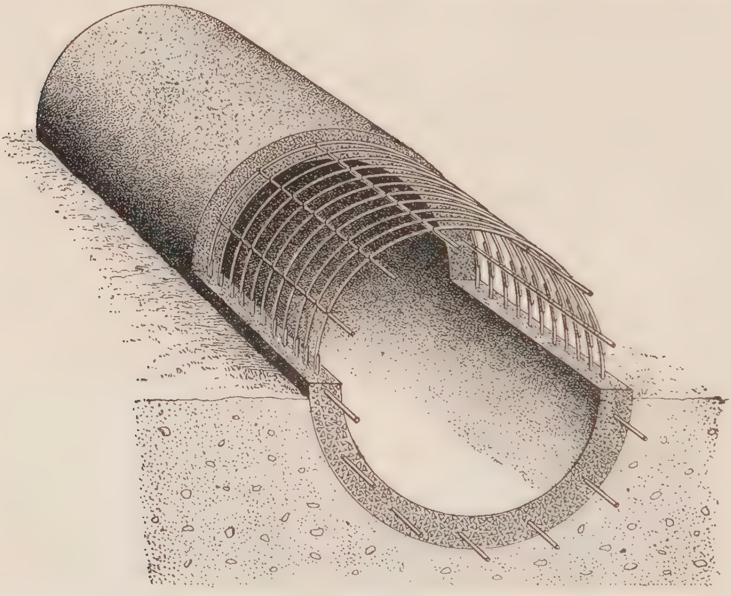


Fig. 43

line. The reinforcement in the sides consists of expanded metal; that at the top consists of expanded metal and rods.

42. Fig. 43 illustrates the method of placing the rods in the cross-section when rods are used for reinforcing a circular sewer. Owing to the difficulty of bending rods to irregular curves, they should not be used for reinforcement in a conduit whose cross-section has an irregular outline.

FORMULAS

RECTANGULAR BEAMS

FUNDAMENTAL PROPOSITIONS

43. Introduction.—The calculations required for the design of reinforced-concrete beams involve a great number of quantities, the relations between which are expressed by complicated formulas requiring tedious work for their use. In practice, various kinds of tables* are used for the purpose of obviating the laborious equations involved. It is necessary, however, to gain a clear conception of the nature of the different elements governing the design, and of their relative importance, in order to be able to use tables or formulas intelligently.

44. Straight-Line Theory.—Many theories have been advanced as a basis for the design of reinforced-concrete beams, and it is not yet known which is most nearly correct. The theory set forth in this Section is known as the **straight-line theory.** The formulas derived from this theory are comparatively simple, are considered to be conservative, and give results that agree closely with those of experiment. They have now been almost universally adopted in this country. Construction according to these formulas is specified by ordinances of many of our large cities, and they have been recommended in the Progress Report of a Joint Committee

*The reinforced-concrete tables in this Section differ from others heretofore published in being available for computations with any specified maximum unit stresses of steel and concrete. These tables, as well as many new formulas and terms here used for the first time, were originated by C. K. Smoley.

composed of members of the American Society of Civil Engineers, American Society for Testing Materials, American Railway Engineering and Maintenance of Way Association, and the Association of American Portland Cement Manufacturers. This committee was organized in 1904 for the purpose of investigating current practice and providing definite information concerning the properties of concrete and reinforced concrete, and to recommend necessary factors and formulas required in the design of structures in which these materials are used. The mentioned Progress Report was published in July, 1909. Many other recommendations of the Joint Committee will be given later.

45. The following assumptions and principles, on which the straight-line theory is based, are practically the same as are used in the Sections on *Strength of Materials* for deducing the theory on which the design of ordinary beams is based, the modifications being due to the properties of the materials forming a reinforced-concrete beam.

1. In general, when a beam is subjected to bending, it is deformed in such a manner that part of the material is compressed, or shortened, and part is extended, or elongated; these parts are separated by a surface called the **neutral surface,** where neither shortening nor elongation takes place. The part of the material that is shortened is under compressional stress and the part that is elongated is under tensional stress. In a reinforced-concrete beam, for reasons given in Art. **7,** the tensional forces of the concrete are neglected, it being assumed that the tension is taken entirely by the steel. Therefore, the material above the neutral surface acts in the same way as in an ordinary beam, and below the neutral surface the only material to be considered is the steel. It is also assumed that the stress in the steel is uniformly distributed over its sectional area, the thickness of the steel being ignored; in other words, the difference in intensity of stress between the upper and the lower fibers of the steel is neglected. When referring to the distance of the steel reinforcement from the neutral axis, the center of gravity of the steel section is meant.

2. In a section of an ordinary beam subjected to bending, there exists a stress couple formed by the resultant of all the compressional stresses on one side and of all the tensional stresses on the other side of the neutral axis. From the foregoing principle it is clear that in a reinforced-concrete beam the stress couple is formed by the total stress in the concrete, on one hand, and the total stress in the steel on the other, and since the forces forming a couple are equal, the total compression must be equal to the total tension; or, denoting the former by T_c and the latter by T_s,

$$T_c' = T_s$$

3. A cross-section of a reinforced-concrete beam subjected to bending is shown in Fig. 44 (a). If the distances x, x', x'',

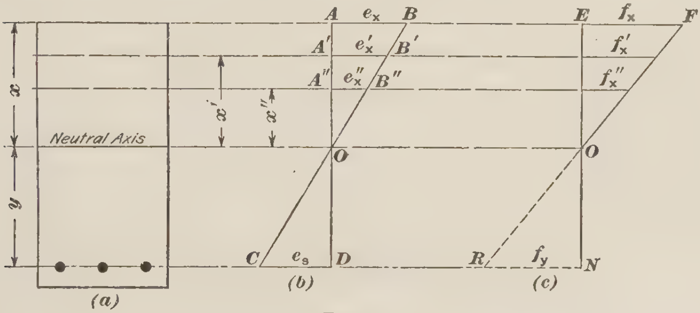


FIG. 44

etc., in (b) are laid off as ordinates (OA, OA', OA'', etc.) and the corresponding unit deformations e_x, e_x', e_x'', etc., at the points A, A', A'', etc., as abscissas (AB, $A'B'$, $A''B''$, etc.), so that $AB = e_x$, $A'B' = e_x'$, etc., the points $B'B'$, etc. will lie in a straight line. Since the triangles ABO, $A'B'O$, etc. are similar, $\frac{e_x}{x} = \frac{e_x'}{x'} = \frac{e_x''}{x''}$, which means that the deformations at different points in a section of a beam are proportional to the respective distances of these points from the neutral axis. In a reinforced-concrete beam this principle applies also to the steel, so that if e_s is the unit elongation of the steel and y its distance from the neutral axis, one may write $\frac{e_s}{y} = \frac{e_x}{x} = \frac{e_x'}{x'}$, etc.

4. The modulus of elasticity of a material has been defined in the Section on *Strength of Materials*, Part 1, as the ratio of the unit stress to the unit deformation. This ratio, for any given material, is assumed to be constant within the elastic limit. If, therefore, f_x, f_x', f_x'', etc., are the unit stresses in the concrete fibers distant x, x', etc., from the neutral axis, f_s the unit stress in the steel, E_c and E_s the moduli of elasticity of concrete and steel, respectively, then, according to this definition, $\frac{f_x}{e_x}=E_c$; $\frac{f_x'}{e_x'}=E_c$, etc. Hence, $f_x=e_x E_c$; $f_x'=e_x' E_c$. Dividing the first of these two equations by the second, gives $\frac{f_x}{f_x'}=\frac{e_x}{e_x'}$; but according to the principle given in paragraph 3, $\frac{e_x}{x}=\frac{e_x'}{x'}$; hence, $\frac{e_x}{e_x'}=\frac{x}{x'}$, and therefore $\frac{f_x}{f_x'}=\frac{x}{x'}$. It follows from this that for one and the same material (in this case the concrete) the intensities of the stresses in different fibers are proportional to their corresponding distances from the neutral axis. In the diagram of Fig. 44 (c) f_x, f_x', etc., are laid off at the corresponding distances x, x' from the neutral axis, and it is evident from the foregoing that FO will also be a straight line. It must not, however, be inferred that, if the line FO is produced to intersect $R\,N$, the line $R\,N$ $(=f_y)$ represents the unit stress in the steel. That this is not so may be seen from the following reasoning: From the definition of the modulus of elasticity, $f_s=e_s E_s$, and if a point is taken in the concrete at the same distance from the neutral axis as the steel, the stress in the concrete at that point will be $f_y=e_y E_c$, in which e_y is the corresponding unit deformation. Dividing the former equation by the latter and noting that according to the principle given in paragraph 3, $e_y=e_s$, there is obtained $\frac{f_s}{f_y}=\frac{E_s}{E_c}$, and if the ratio $\frac{E_s}{E_c}$ is denoted by n, $\frac{f_s}{f_y}=n$; hence, $f_s=n\,f_y$. *The unit stress in the steel is, therefore, n times as great as that in a fiber of concrete located at the same distance from the neutral axis as the steel.*

46. Position of Neutral Surface.—Let x, Fig. 45, be the distance of the top $C D$ of a reinforced-concrete beam from the neutral axis $B E$; A, the total area of steel reinforcement; b, the width of the beam; and d, the distance of the steel from the top of the beam. This distance d is usually called the **effective depth** of the beam, and the area $b d$ the **effective area** of the section. A, the area of the steel, is usually expressed as a fractional part of $b d$ by introducing a coefficient $p = \frac{A}{b d}$, the ratio of the area of the steel to that of the concrete. Hence, $A = p b d$. If, for example, $b = 10$ inches, $d = 15$ inches, and $p = .005$ the area of steel used is $.005 \times 10 \times 15 = .75$ square inch. The area of the steel is sometimes also expressed as a percentage of $b d$, which is equal to $100 p$. For instance, in the example just given, the area of the steel used is $\frac{1}{2}$ per cent. of $b d$. It is also convenient to express x as a fractional part of d by denoting the ratio $\frac{x}{d}$ by k; hence, $x = k d$.

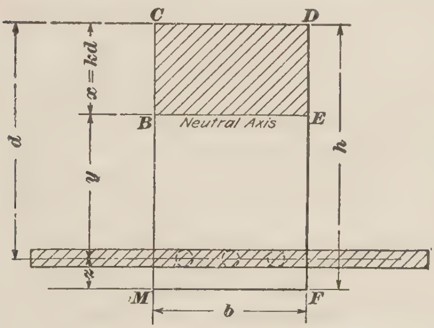


FIG. 45

The method of determining the position of the neutral surface will now be shown. In a homogeneous beam the neutral axis passes through the center of gravity of the section. A section of a reinforced-concrete beam may be treated as a homogeneous section, if the steel is considered to be replaced by a narrow strip of concrete having an area just sufficient to produce the same effect as the steel in resisting bending moment. The section may then be treated as if it consisted of the shaded areas, Fig. 45, and is called a **transformed section.** If A_q is the equivalent area of concrete substituted for the steel, and, as before, f_s is the intensity of stress in the steel and f_y the intensity of stress in the concrete at the distance y from the neutral axis; then, since the total stress in the steel is $T_s = A f_s$, the area A_q must be such as to satisfy the condition

$A_q f_y = A f_s$. Substituting for f_s its value derived at the end of Art. **45,** $A_q f_y = A n f_y$; hence,

$$A_q = A n \qquad (1)$$

If, therefore, it is conceived that the steel is replaced by a strip of concrete having the area $A n$, the section becomes homogeneous, and its neutral axis must pass through its center of gravity. To find its position, take moments about the line CD, remembering that the area $MBEF$ is disregarded. The moment of $BCDE$ is $x b \times \frac{x}{2} = \frac{x^2 b}{2}$; the moment of the equivalent area substituted for the steel is $A n \times d$. The sum of these moments must be equal to the product of the total area $x b + A n$ by x, the distance of the neutral axis from CD. Therefore,

$$\frac{x^2 b}{2} + A n d = (x b + A n) x$$

Substituting for A its value $p b d$, and solving for x,

$$x = d\left(\sqrt{p^2 n^2 + 2 p n} - p n\right);$$

hence,

$$\frac{x}{d} = k = \sqrt{p^2 n^2 + 2 p n} - p n \qquad (2)$$

This formula shows that the position of the neutral axis depends only on p and n. As will be remembered, $p = \frac{A}{b d}$ and $n = \frac{E_s}{E_c}$. E_s has an average value of 30,000,000 pounds per square inch, but E_c varies greatly with the mixture of the concrete, its age, and the workmanship. The value of n is usually specified in building ordinances. For the values of n commonly used in practice, Table I, which appears at the end of this Section, gives the values of k by intervals of .0002 from $p = .0002$ to $p = .02$.

EXAMPLE.—In a section of a reinforced-concrete beam let $b = 12$ inches, $d = 15$ inches, and $A = 2.16$ square inches. Find the position of the neutral axis for $n = 12$.

SOLUTION.—Here $p=\frac{2.16}{12\times15}=.012$. Substituting known values in formula 2,

$$k=\sqrt{.012^2\times12^2+2\times.012\times12}-.012\times12=.4116$$

In Table I for $n=12$, this value of k may be found directly in the same horizontal line with $p=.012$. The distance of the neutral axis from the top of the beam is therefore $x=k\,d=.4116\times15=6.17$ in. Ans.

It is well to note in Table I that k increases with p, the amount of steel used.

EXAMPLES FOR PRACTICE

1. If $b=18$ inches, $d=36$ inches, $A=3.24$ square inches, and $n=15$, find, by formula 2, Art. 46, the distance from the top of the beam to its neutral axis. Ans. $11\frac{1}{2}$ in.

2. In a certain beam $b=15$ inches, $d=24$ inches, $A=3.6$ square inches, and $n=15$. Find the value of k by use of Table I. Ans. $k=.4179$

47. Total Stress in Concrete and the Center of Compression.—It has been explained in Art. 45 that the total stress in the steel and the total stress in the concrete form a couple, and an expression has been found for the total stress in the steel, which is

$$T_s=A\,f_s=p\,b\,d\,f_s \qquad (1)$$

To find an expression for the total stress in the concrete, consider the beam section shown in Fig. 46. In (b), f_c is the

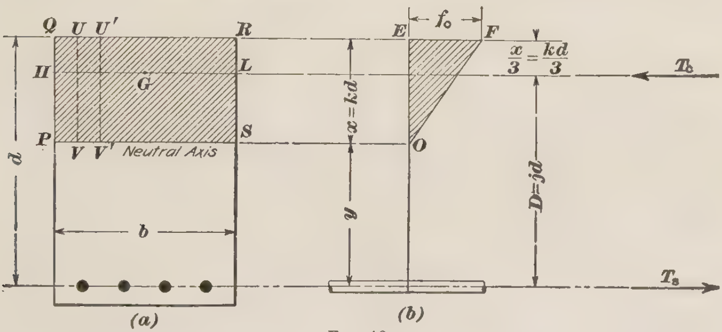


FIG. 46

intensity of stress in the outermost fibers. Since, by the principle given in paragraph 3, Art. 45, $F\,O$ is a straight line,

the average intensity of stress, that is, the average stress per unit of area, is $\frac{f_c}{2}$, and the total stress on the area $b\,x$ is

$$T_c = \frac{f_c\,b\,x}{2} = \frac{f_c\,k\,b\,d}{2} \qquad (2)$$

To find the distance of this resultant force from the top of the beam, consider an area $P\,Q\,U\,V$ in which $Q\,U = P\,V = 1$. The total stress on this area, in which b equals unity, is, by formula **2**, $\frac{f_c\,x}{2}$. But this expression also represents the area of the triangle $E\,O\,F$; therefore, the force acting on the area $P\,Q\,U\,V$ may be considered as concentrated at the center of gravity of this triangle, which is distant $\frac{x}{3}$ from $E\,F$. To get the expression for T_c, the force on the total area $P\,Q\,R\,S$, $\frac{f_c\,x}{2}$ must be multiplied by b, which means the adding to $P\,Q\,U\,V$ of as many other rectangles, such as $U\,V\,U'\,V'$, in which $U\,U'$ = unity, as there are units in b. Since, in each of these areas, the center of pressure is on the line $H\,L$, the point of application G of the resultant pressure must also be on this line. Hence, the distance of T_c from the top of the beam is $\frac{x}{3}$, and from the center of the steel is

$$D = d - \frac{x}{3} = d - \frac{k\,d}{3} = d\left(1 - \frac{k}{3}\right) \qquad (3)$$

This distance D is the lever arm of the stress couple, and is usually expressed as a fractional part of d, so that $D = j\,d$, where $j = \frac{D}{d}$. Combining this with formula **3**,

$$j = 1 - \frac{k}{3} \qquad (4)$$

The value of j is also tabulated in Table I for each value of p therein contained.

EXAMPLE.—What is the value of j in the example of Art. **46**?

SOLUTION.—Substituting in formula **4** for k the value .4116 found in that example, $j=1-\frac{.4116}{3}=.8628$. Ans.

In Table I for $n=12$, this value of j may be found directly in the same line with $p=.012$ and $k=4116$.

EXAMPLES FOR PRACTICE

1. In a certain beam $b=10$ inches, $d=20$ inches, $A=1.2$ square inches, and n is taken as 15. Find by the formulas in Arts. **46** and **47** the value of j. Ans. $j=.8854$

2. In a certain beam $b=12$ inches, $d=20$ inches, $A=1.68$ square inches, and n is taken as 12. Find the length in inches of the arm of the resisting couple by the use of Table I. Ans. 17.77 in.

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48. Working Resisting Moment.—The bending moment, that is, the moment of the external forces acting on a beam, is resisted by the moment of the stress couple, which is the resisting moment of the beam. Since $T_c=T_s$, if M denotes the bending moment, $M=T_c\,j\,d$, and also $M=T_s\,j\,d$, or, using the values of T_c and T_s previously derived,

$$M=\frac{k\,j}{2}\,b\,d^2 f_c \qquad (1)$$

and

$$M=p\,j\,b\,d^2 f_s \qquad (2)$$

Letting $\frac{k\,j}{2}=C_c$, and $p\,j=C_s$ and substituting, formulas **1** and **2** become

$$M=C_c\,b\,d^2 f_c \qquad (3)$$

and

$$M=C_s\,b\,d^2 f_s \qquad (4)$$

Formulas **3** and **4** have the same form as the formula for the bending moment of an ordinary beam, which is $M=\frac{1}{6}\,b\,d^2 f$, C_c and C_s corresponding to the coefficient $\frac{1}{6}$. As is well known, the expression $\frac{1}{6}\,b\,d^2$ is called the section modulus of the beam. In this Section, $C_c\,b\,d^2$ and $C_s\,b\,d^2$ will, accordingly, be called the **section moduli for concrete and steel**, respectively, and C_c and C_s the **section moduli coefficients for concrete and steel**, respectively. It must be kept in mind that these coefficients are variable, depending on p, or the amount

of steel used. C_c and C_s are tabulated in Table I for each value of p therein given.

EXAMPLE.—Find: (*a*) the section moduli coefficients and (*b*) section moduli for steel and concrete for the section given in Art. **46**.

SOLUTION.—(*a*) Here $p=.012$, $k=.4116$, $j=.8628$, $b=12$, and $d=15$.

Therefore, $$C_c=\frac{.4116\times.8628}{2}=.1776. \quad \text{Ans.}$$

$$C_s=.012\times.8628=.01035. \quad \text{Ans.}$$

These values of C_c and C_s will be found in Table I for $n=12$, opposite $p=.012$.

(*b*) Since $b\,d^2=12\times15^2=2{,}700$, the section modulus for concrete is $.1776\times2{,}700=479.5$. Ans.

Section modulus for steel is $.01035\times2{,}700=27.95$. Ans.

Equations **3** and **4** furnish the fundamental formulas for designing and investigating reinforced-concrete beams; their analysis and practical application, however, will be shown to better advantage in subsequent articles after certain additional principles have been introduced.

Formula **4** can sometimes be used with advantage in the following modified form:

Since $T_s=A\,f_s$ and $M=T_s\,j\,d$,

$$M=A\,j\,d\,f_s \qquad (5)$$

In this formula the section modulus is $A\,j\,d$.

EXAMPLES FOR PRACTICE

1. Calculate the section moduli coefficients for a beam for which $p=.01$, $j=.8607$, and $k=.4179$.

Ans. $\begin{cases} C_s=.008607 \\ C_c=.1798 \end{cases}$

2. Calculate the section moduli for steel and concrete for a beam when $p=.007$, $j=.8783$, $k=.3651$, $b=14$ inches, and $d=20$ inches.

Ans. $\begin{cases} 897.7 \text{ in.}^3 \text{ for concrete} \\ 34.4 \text{ in.}^3 \text{ for steel} \end{cases}$

3. Find from Table I the section moduli coefficients for a beam when $p=.008$ and $n=15$.

Ans. $\begin{cases} C_s=.006975 \\ C_c=.1676 \end{cases}$

49. Economic Ratio of Steel.—The economic design of a structure requires that all parts have a uniform degree, or factor, of safety. If, for instance, in a steel structure that is specified to carry 15,000 pounds per square inch, some members are developing only 10,000 pounds per square inch, the superfluous material is not only a waste but adds a dead load that has to be carried by the structure. As there are two materials in a reinforced-concrete beam, the principle of economic design requires that they both develop their maximum allowable unit stresses under the same load. Since, as the subsequent analysis will show the relative stresses on the two materials depend only on p, the principle just stated requires that the two materials be employed in a definite proportion. If not enough steel is used, this material will reach its maximum allowable stress first, and the beam cannot be loaded further so as to develop the allowable unit stress in the concrete, because that would overstress the steel. On the other hand, if too much steel is used, the strength of the beam will be limited by the concrete, which would develop its full strength before the full strength of the steel could be brought into play. That steel ratio at which both materials can be stressed under the same load to their specified unit stresses is called the **economic steel ratio**, and will be designated by p_e. It is also called sometimes the **economic percentage** of steel, meaning then 100 p_e. Sometimes it is also referred to as the **critical value** of steel. It must be borne in mind that the economic ratio of steel is a relative term referring to definite allowable unit stresses. As the latter vary greatly, it cannot be fixed once for all.

50. Position of Neutral Axis for Economic Ratio of Steel.—The ratio $\frac{f_s}{f_c}$, of the unit stresses actually produced in a reinforced-concrete beam under any loading and corresponding to any value of p, will be here called simply the **stress ratio**, and designated by r. In order to locate the neutral axis when the economic ratio of steel is used, an algebraic expression for k in terms of r will first be deduced.

By the principle explained in paragraph 4 of Art. **45**,

$$f_s = e_s E_s$$

and

$$f_c = e_c E_c$$

Dividing the former equation by the latter, $r = \frac{f_s}{f_c} = \frac{e_s \times E_s}{e_c \times E_c}$; or, using the previous notation, $r = \frac{e_s}{e_c} \times n$. Noting that by principle in paragraph 3 of Art. **45**, $\frac{e_s}{e_c} = \frac{y}{x}$, and substituting, $r = \frac{y}{x} \times n$. Transposing, $\frac{x}{y} = \frac{n}{r}$; hence,* $\frac{x}{x+y} = \frac{n}{n+r}$. But by the relations established in Art. **46**, $\frac{x}{x+y} = \frac{x}{d} = k$.

Therefore,

$$k = \frac{n}{n+r} \qquad (1)$$

This formula gives k in terms of r if n is considered as a constant. Although very simple, this formula is of no general practical value in determining the position of the neutral axis; because f_s and f_c, and consequently r, cannot be known, generally, before k is determined. The formula can, however, be applied with advantage for the case when the economic ratio of steel is used, because in that case r is the ratio of the specified unit stresses and is known beforehand.

NOTE.—To avoid confusion and repetitions, all the values corresponding to p_e, and derived from it will hereafter be distinguished by the usual letter, to which the subscript e will be added.

Applying formula **1** to the case when the economic ratio of steel is used

$$k_e = \frac{n}{n+r_e} \qquad (2)$$

EXAMPLE.—Find the location of the neutral axis in a beam in which $d = 18$ inches, the maximum unit stresses being 15,000 pounds per square

*If four quantities are in proportion, the first is to the sum of the first and second as the third is to the sum of the third and fourth. Thus, in the proportion $3 : 5 = 7 : x$, $3 : (3+5) = 7 : (7+x.)$ From either proportion, $x = 11\frac{2}{3}$. Hence, if $\frac{x}{y} = \frac{n}{r}$, $\frac{x}{x+y} = \frac{n}{n+r}$.

inch for steel and 500 pounds per square inch for concrete, when the economic ratio of steel is used. Use $n=15$.

SOLUTION.—To apply formula **2**, substitute for r_e the value $\frac{15{,}000}{500}=30$. Then, $k_e=\frac{15}{15+30}=\frac{1}{3}$. Therefore, the distance of the neutral axis from the top of the beam is

$$x_e=k_e\,d=\tfrac{1}{3}\times 18=6 \text{ in.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. Find the neutral axis in a beam for which $b=10$ inches and $d=16$ inches when the economic ratio of steel is used. The maximum tensile stress in the steel is to be 16,000 pounds per square inch, and the maximum compressive stress in the concrete is to be 650 pounds per square inch. It is assumed that $n=15$. Ans. 6.06 in. from top

2. In a certain beam the stress in the concrete is to be 500 pounds per square inch and the tensile stress in the steel is to be 12,000 pounds per square inch. If $n=15$, what is the value of k_e? Ans. $k_e=.3846$

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51. Determination of Economic Steel Ratio.—Formula **2** of the preceding article affords an indirect method of determining the economic steel ratio by means of Table I, because when k_e has been found the corresponding value of p in the table will be the economic ratio. For instance, in the example of the preceding article the nearest value of p corresponding to $k=.333$ is given in Table I as .0056, which is the economic steel ratio. It is, however, desirable to have a direct formula for this purpose. Referring to formulas **3** and **4**, Art. **48**, which are

$$M=C_c\,b\,d^2 f_c, \text{ and } M=C_s\,b\,d^2 f_s,$$

f_c and f_s are here the stresses produced by the external moment M. Since M is the same in both equations, it follows that $C_c\,b\,d^2 f_c=C_s\,b\,d^2 f_s$, hence $\frac{f_s}{f_c}=\frac{C_c}{C_s}$, which shows that the stress ratio equals the inversed ratio of the section moduli coefficients, or

$$r=\frac{C_c}{C_s} \qquad (1)$$

Putting in this equation for C_c and C_s their values $\frac{kj}{2}$ and pj, respectively, and canceling,

$$r=\frac{k}{2p} \qquad (2)$$

Solving for p,

$$p=\frac{k}{2r} \qquad (3)$$

Substituting for k its value in formula **1**, Art. **50**,

$$p=\frac{n}{2r(n+r)} \qquad (4)$$

Applying this formula for the case of the economic ratio of steel where r is given,

$$p_e=\frac{n}{2r_e(n+r_e)} \qquad (5)$$

EXAMPLE.—Find the economic ratio of steel for the maximum allowable unit stress of 12,000 pounds per square inch for steel and of 500 pounds per square inch for concrete, when $n=8$.

SOLUTION.—Applying formula **5**, and substituting $n=8$ and $r_e=\frac{12{,}000}{500}=24$,

$$p_e=\frac{8}{2\times 24\times(8+24)}=.0052. \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. Find the economic ratio of steel for the maximum allowable unit stresses of 16,000 pounds per square inch for steel and of 650 pounds per square inch for concrete, when $n=15$. Ans. .00769

2. Find the critical value of p when $F_s=15{,}000$, $F_c=500$, and $n=15$. Ans. .00556

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52. Economic Steel Ratio by Table I.—To obviate the use of formulas for the computation of p_e, the columns headed $r=\frac{f_s}{f_c}$ of Table I have been provided. They give the stress ratios as calculated by formula **2** of the preceding article for the corresponding values of p, and can be made the basis

for reading the value of p from a given r. Since, for the critical value of p, the stress ratio coincides with the ratio of the maximum allowable unit stresses r_e, when a value in the column of r of Table I is found that is equal, or nearly equal, to r_e, the corresponding value in the column of p is equal, or nearly equal, to p_e. To find, for instance, the economic steel ratio in the example of the preceding article, look in Table I for $n=8$, in the column r, for the number $\frac{12{,}000}{500}=24$, and the corresponding p will be found as .0052.

PRACTICAL APPLICATION

53. As stated before, formulas **3** and **4**, Art. **48**, which are $M=C_s\,b\,d^2 f_s$ and $M=C_c\,b\,d^2 f_c$ furnish the fundamental equations for designing and investigating rectangular reinforced-concrete beams. Their application will now be shown. It will be remembered that in these formulas f_s and f_c are the actual stresses induced in the beam by the bending moment M. In practice, these stresses are rarely required, but if needed, these formulas are available for their determination. It is of far greater importance, however, to express these formulas in terms of the maximum allowable unit stresses. Let these unit stresses be denoted by F_s and F_c, which may or may not coincide with f_s and f_c. Now, as has already been explained, when the economic steel ratio is used, both the concrete and the steel reach the maximum allowable unit stresses under the same load, while if any other than the economic ratio is used, one of the materials will be stressed to its limit under a smaller load than the other. If therefore, F_s and F_c are substituted in the formulas under consideration, the resulting M will be the same in both formulas only when the economic steel ratio p_e is used. For any other proportion of steel the results will not be equal; and if designated by M_s and M_c, respectively, the formulas become

$$M_s=C_s\,b\,d^2F_s \qquad (1)$$

$$M_c=C_c\,b\,d^2F_c \qquad (2)$$

If the steel ratio is less than p_e, F_s will be reached under a smaller load than F_c, consequently M_s is then smaller than M_c, and to assure the safety of the design, formula **1** must be employed; if the steel ratio exceeds p_e, formula **2** is used.

It is obvious that when the economic steel ratio is used $M_s = M_c$. In this case either formula will give the same result and one may be employed to verify the other.

It must be kept in mind, that in formulas **3** and **4**, Art. **48**, M designates the bending moment, that is, the moment of the external forces; it must therefore be one and the same in both formulas, while in formulas **1** and **2** of this article M_s and M_c are the *resisting moments* of the steel and concrete, respectively. This is the reason why they may differ from each other. When applying the latter formulas to practical problems, the bending moment is equated to M_s or M_c according to which is the smaller.

PRACTICAL EXAMPLES

54. Table I.—In the foregoing all the principles and formulas required for use in designing and calculating reinforced-concrete rectangular beams have been provided. As has already been demonstrated, the work is greatly facilitated by the use of Table I. Its chief advantage is in its being available for any specified unit stresses. It has been compiled to four significant figures, but three and in a great many cases two significant figures will be sufficient. As it is well to have an idea of the accuracy of the results obtained by the use of these tables it may be stated that the maximum errors when four, three, or two significant figures are used are, respectively, $\frac{1}{20}$ per cent., $\frac{1}{2}$ per cent., and 5 per cent. of the true results. In the following examples three significant figures will be used in most cases.

55. To design a Beam.—When a beam is to be designed, the dimensions of the section and the amount of steel will be the unknown quantities, the other data being given. Three cases will be considered.

Case I.—Given M, n, F_s, F_c, required b, d, and the amount of steel to be used. This is the case most frequently occurring in practice. As the percentage of steel is not specified, the economic ratio will be used and either formula **1** or **2**, Art. **53**, applied, for which purpose C_s or C_c will be needed. If Table I is used, the latter are taken directly as the values corresponding to $r_e = \frac{F_s}{F_c}$. If formulas are employed, the procedure is as follows: First calculate k_e from formula **2**, Art. **50**, which is $k_e = \frac{n}{n+r_e}$; then by formula **3**, Art. **51**, $p_e = \frac{k_e}{2\,r_e}$; next, by formula **4**, Art. **47**, $j = 1 - \frac{k_e}{3}$; finally from the equations of Art. **48**, $C_s = p_e\,j_e$, and $C_c = \frac{k_e\,j_e}{2}$.

When C_s and C_c have been computed, formulas **1** and **2**, Art. **53**, can be applied. As all quantities except b and d will then be known, the problem is solved essentially in the same way as in the case of an ordinary beam; namely, one dimension is assumed and the equation solved for the other. If b is assumed, then

$$d = \sqrt{\frac{M_s}{C_s\,b\,F_s}} \qquad (1)$$

or,

$$d = \sqrt{\frac{M_c}{C_c\,b F_c}} \qquad (2)$$

If d is assumed, the formula for b is

$$b = \frac{M_s}{C_s\,d^2\,F_s} \qquad (3)$$

or,

$$b = \frac{M_c}{C_c\,d^2\,F_c} \qquad (4)$$

The economic ratio of b to d depends on the percentage of steel used, the ratio of cost of steel to cost of concrete per unit of volume, and other factors, and cannot be expressed in a formula that would be convenient for practical use. In practice, the

conditions of the design will usually determine one or the other dimension. When this is not the case some engineers make b about one-half of d. When b is small in comparison with d, there is danger of the beam failing by diagonal tension, as explained in a subsequent article.

If b is made equal to $\frac{d}{2}$, the preceding formulas become, respectively:

$$d=\sqrt[3]{\frac{2\,M_s}{C_s F_s}} \qquad (5)$$

or,

$$d=\sqrt[3]{\frac{2\,M_c}{C_c F_c}} \qquad (6)$$

and

$$b=\frac{d}{2} \qquad (7)$$

When b and d have been computed, the amount of steel A is, from the equation of Art. **46**, $A=p_e\,b\,d$.

EXAMPLE.—Design a reinforced-concrete beam to resist a total maximum bending moment of 2,000,000 inch-pounds, taking $F_s=16{,}000$ pounds per square inch, $F_c=600$ pounds per square inch, and $n=15$.

SOLUTION.—In this case, $r_e=\frac{F_s}{F_c}=\frac{16{,}000}{600}=26.67$. Then, by formula **2**, Art. **50**, $k_e=\frac{n}{n+r_e}=\frac{15}{15+26.67}=.360$; by formula **3**, Art. **51**, $p_e=\frac{k_e}{2\,r_e}=\frac{.360}{2\times 26.67}=.00675$; by formula **4**, Art. **47**, $j_e=1-\frac{k_e}{3}=1-\frac{.360}{3}=.88$. Then, substituting these values in the equations of Art. **48** for C_s and C_c, $C_s=p_e\,j_e=.00675\times.88=.00594$; and $C_c=\frac{k_e\,j_e}{2}=\frac{.36\times.88}{2}=.1584$.

The values of C_s and C_c corresponding to r_e could have been taken directly from Table I for $n=15$. Here, $r_e=26.67$, and the nearest value of r in the table is 26.55. Looking in the same horizontal line it is found that $C_s=.00598$, and $C_c=.159$, to three significant figures. It will usually be sufficiently accurate to take from the table the values of C_s and C_c corresponding to the value of r that is nearest the calculated value of r_e. For greater accuracy in important work, however, values of C_s and C_c corresponding to intermediate values of r can be determined from the table by interpolation. In this case, it is found by interpolation that $C_s=.00594$, and $C_c=.1584$, as calculated.

One dimension of the beam is now assumed, and the corresponding formula of this article applied for the other dimension. Taking $b = 20$ inches, and applying formula **1**,

$$d = \sqrt{\frac{M_s}{C_s\, b\, F_s}} = \sqrt{\frac{2{,}000{,}000}{.00594 \times 20 \times 16{,}000}} = 32.4 \text{ in.}$$

Using formula **2** as a check, $d = \sqrt{\frac{M_c}{C_c\, b\, F_c}} = \sqrt{\frac{2{,}000{,}000}{.1584 \times 20 \times 600}} = 32.4$ in., as before.

For the area of steel, apply the equation of Art. **46**, $A = p_e\, b\, d = .00675 \times 20 \times 32.4 = 4.37$ sq. in.

It must be borne in mind that, as has been established in Art. **46**, d signifies the effective depth of the beam, meaning the distance of the center of the steel from the top of the beam. To find the total depth h, Fig. 45, a certain amount z must be added to d, which varies from about 1 to 2 inches, according to the character and size of the beam. This matter will be discussed in detail later.

EXAMPLES FOR PRACTICE

1. Design a beam by use of the formulas to resist a total moment of 1,000,000 inch-pounds. F_s must not exceed 15,000 pounds per square inch and F_c must not exceed 500 pounds per square inch; n is taken at 15; and it is assumed that $b = 15$ inches.

Ans. $\begin{cases} d = 30 \text{ in.} \\ A = 2.5 \text{ sq. in.} \end{cases}$

2. Design, by using Table I, a beam to carry a total load, including its own weight, of 1,000 pounds per foot over a span of 24 feet. Use $F_s = 12{,}000$ pounds per square inch, $F_c = 400$ pounds per square inch, and $n = 18$. Assume that $d = 30$.

Ans. $\begin{cases} b = 14.7 \text{ in.} \\ A = 2.73 \text{ sq. in.} \end{cases}$

Case II.—In the preceding case in designing a reinforced-concrete beam, the critical value of steel was used. The designer will always do so whenever he is free to select the steel ratio. Conditions, however, often occur necessitating the use of other than the economic steel ratio. As explained before, when the steel ratio is less than p_e, the concrete will not develop its full strength without overstressing the steel, and when the steel ratio is greater than p_e, the full strength of the steel cannot be utilized without overstressing the con-

crete. Since the steel is a material that can be more relied on and is also more expensive than concrete, it is desirable to develop the full strength of the steel rather than that of the concrete. It is for this reason that good designers avoid using steel above the economic percentage. Sometimes, however, it cannot be avoided.

As in Case I, let M, n, F_s, F_c be given and let it be required to design the beam with any value of p. It is clear from the foregoing that when p is less than p_e the strength of the steel will govern the design, and the beam must be designed by formulas **1** and **3**; whereas, when p is greater than p_e, the design will depend upon the concrete, and formulas **2** and **4** will have to be applied. The procedure is, therefore, as follows: First determine $r_e=\frac{F_s}{F_c}$, then obtain p_e either from Table I or by formula **5**, Art. **51**, which is $p_e=\frac{n}{2\,r_e\,(n+r_e)}$. Then, if p is less than p_e, take from the table, or calculate by the method given in Art. **48**, the value of C_s and use formulas **1** and **3**. When p is greater than p_e, C_c is determined and formulas **2** and **4** applied.

Example 1.—A certain beam must resist a bending moment of 500,000 inch pounds. It is decided to make $p=.004$, $F_s=17{,}000$ and $F_c=600$ pounds per square inch, $n=12$, and $d=22$. Find b and A.

Solution.—Here $r_e=\frac{17{,}000}{600}=28.33$. Then, applying formula **5**, Art. **51**, $p_e=\frac{n}{2\,r_e\,(n+r_e)}=\frac{12}{2\times 28.33\times(12+28.33)}=.0053$. Or, using Table I for $n=12$, the nearest value of r is 28.49, and the corresponding value of p is .0052. The given value $p=.004$ is less than the economic percentage, and the steel therefore governs the design; that is, formula **3** must be applied to determine the value of b. From formula **2**, Art. **46**, $k=\sqrt{p^2\,n^2+2\,p\,n}-p\,n=\sqrt{.004^2\times 12^2+2\times .004\times 12}-.004\times 12=.266$. From formula **4**, Art. **47**, $j=1-\frac{k}{3}=1-\frac{.266}{3}=.9113$. Then, from the equation of Art. **48**, $C_s=p\,j=.004\times .9113=.003645$.

In Table I for $n=12$, opposite $p=.004$, it is found that $C_s=.003646$. Now, applying formula **3**, $b=\frac{500{,}000}{.00365\times 22^2\times 17{,}000}=16.6$ in. Applying the equation of Art. **46**, $A=p\,b\,d=.004\times 16.6\times 22=1.46$ sq. in.

EXAMPLE 2.—Design a beam to carry a load, including its own weight, of 1,800 pounds per foot, over a span of 18 feet. The following values are to be used: $F_s=18{,}000$, $F_c=700$, $p=.008$, and $n=10$.

SOLUTION.—First find r_e and p_e in order to determine whether or not the percentage of steel is above the economic percentage; $r_e=\frac{18{,}000}{700}=25.71$. Then, $p_e=\frac{n}{2\,r_e\,(n+r_e)}=\frac{10}{2\times 25.71\times(10+25.71)}=.00545$. Or, in Table I for $n=10$, the value of r nearest to r_e is 25.84, and the corresponding value of p is .0054. As the value of p is greater than the economic percentage, the concrete governs the design, and formula **2** or formula **4** must be used. Proceeding as before, $k=\sqrt{.008^2\times 10^2+2\times .008\times 10}-.008\times 10=.328$. Then, $j=1-\frac{.328}{3}=.891$. From the equation of Art. **48**, $C_c=\frac{k\,j}{2}=\frac{.328\times .891}{2}=.146$. Or, in Table I for $n=10$, opposite $p=.008$, $C_c=.146$.

Assuming $b=16$ in., d is obtained by means of formula **2**. Here, $M_c=\frac{1{,}800\times 18^2}{8}=72{,}900$ ft.-lb. $=874{,}800$ in.-lb. Then,

$$d=\sqrt{\frac{874{,}800}{.146\times 16\times 700}}=23 \text{ in.}$$

For the amount of steel,

$$A=p\,b\,d=.008\times 16\times 23=2.94 \text{ sq. in.}$$

EXAMPLES FOR PRACTICE

1. In designing a beam take $p=.006$, $F_s=12{,}500$; $F_c=450$, $n=20$, $M=700{,}000$ inch-pounds, and $d=25$ inches. Find b and A.

$$\text{Ans.}\begin{cases} b=17.2 \text{ in.} \\ A=2.55 \text{ sq. in.}\end{cases}$$

2. In a certain beam p is to be taken as .009, $F_s=15{,}500$, $F_c=550$, $n=15$, and $b=12$ inches. Determine d and A, when $M=350{,}000$ inch-pounds.

$$\text{Ans.}\begin{cases} d=17.5 \text{ in.} \\ A=1.89 \text{ sq. in.}\end{cases}$$

Case III.—It sometimes occurs that the dimensions of a beam section, b and d, are fixed, and it is required to find the amount of steel to be used for a certain bending moment M, when n, F_s, and F_c are also given. The solution of this problem by formulas is quite complicated but it is readily solved by

means of Table I. Since it cannot be foreseen whether the amount of steel required is above or below the economic ratio, it is not known which of the fundamental formulas $M_s = C_s\, b\, d^2 F_s$ or $M_c = C_c\, b\, d^2 F_c$ governs the design. When Table I is used it is most expeditious, in this case, to determine p by the aid of both formulas and adopt the greater quantity. Substituting for M_s and M_c the known bending moment M, and solving for C_s and C_c,

$$C_s = \frac{M}{b\, d^2 F_s} \qquad (8)$$

and

$$C_c = \frac{M}{b\, d^2 F_c} \qquad (9)$$

Now, referring to Table I, there will be found a value of p corresponding to each coefficient, of which the greater should be selected.

EXAMPLE.—A beam is to resist a bending moment of 800,000 inch-pounds. If $b = 18$ inches, $d = 27$ inches, $F_s = 12{,}500$, $F_c = 500$, and $n = 15$, what is the amount of steel to be used?

SOLUTION.—Substituting the known values in formula 8,

$$C_s = \frac{M}{b\, d^2\, F_s} = \frac{800{,}000}{18 \times 27^2 \times 12{,}500} = .00488$$

From formula 9,

$$C_c = \frac{M}{b\, d^2\, F_c} = \frac{800{,}000}{18 \times 27^2 \times 500} = .122$$

In Table I for $n = 15$, it is found that for $C_s = .00488$ the value of p is between .0054 and .0056, and for $C_c = .122$, p is between .0032 and .0034. The value corresponding to C_s, being the greater, should therefore be used, and by interpolation $p = .0055$. The required amount of steel is, therefore, $A = p\, b\, d = .0055 \times 18 \times 27 = 2.67$ sq. in. Ans.

When convenient tables are not at hand, the following method by formulas may be employed: First determine the bending moment that the beam is capable of resisting if the economic percentage of steel were used. Let that moment be denoted by M_e. If M_e is less than the given bending moment M, this will show that the amount of steel required is greater than the economic ratio; consequently, the concrete governs the design. Therefore, having determined the value of C_c by

formula **9**, the exact amount of steel must be such as to satisfy the condition $C_c=\frac{j\,k}{2}$, or $C_c=\left(1-\frac{k}{3}\right)\frac{k}{2}$. Solving for k,

$$k=\frac{3-\sqrt{9-24\,C_c}}{2} \qquad \textbf{(10)}$$

Formula **10** gives the exact position of the neutral axis of the beam, and the steel ratio p must be such as to insure this value of k. From formula **2**, Art. **46**, $k=\sqrt{p^2\,n^2+2\,p\,n}-p\,n$. Solving for p, there is obtained finally,

$$p=\frac{k^2}{2\,n\,(1-k)} \qquad \textbf{(11)}$$

Knowing p, the area of steel is $A=p\,b\,d$.

EXAMPLE.—Assume the same conditions as in preceding example, $b=18$, $d=27$, $F_s=12{,}500$, $F_c=500$, and $n=15$, but take $M=1{,}200{,}000$ inch-pounds, and determine by the formulas the value of A.

SOLUTION.—The economic percentage of steel must first be calculated. Here, $r_e=\frac{12{,}500}{500}=25$. Then, from formula **2**, Art. **50**, $k_e=\frac{n}{n+r_e}$ $=\frac{15}{15+25}=.375$, and from formula **3**, Art. **51**, $p_e=\frac{k_e}{2\,r_e}=\frac{.375}{2\times 25}=.0075$. Then, $j_e=1-\frac{k_e}{3}=1-\frac{.375}{3}=.875$. Applying formula **2**, Art. **48**, $M_e=p_e\,j_e\,b\,d^2\,F_s=.0075\times .875\times 18\times 27^2\times 12{,}500=1{,}076{,}000$ in.-lb.

This is less than the bending moment $M=1{,}200{,}000$ in.-lb, and the amount of steel required is therefore greater than the economic percentage; that is, the concrete governs the design. From formula **9**, $C_c=\frac{M}{b\,d^2\,F_c}=\frac{1{,}200{,}000}{18\times 27^2\times 500}=.183$. Then by formula **10**, $k=\frac{3-\sqrt{9-24\times .183}}{2}=.427$, and from formula **11**, $p=\frac{.427^2}{2\times 15\times (1-.427)}$ $=.0106$. Hence, the required area of steel is $A=p\,b\,d=.0106\times 18\times 27$ $=5.15$ sq. in.

If M_e is greater than M; that is, when the beam is capable of carrying more than M, when the economic ratio of steel is used, this will show that the amount of steel required is less than the critical value. The strength of the beam is then dependent on the steel and the exact steel ratio p may be

determined from the relation given in Art. **48,** $C_s = p\,j$, where C_s is the value of the section modulus coefficient as determined from formula **8.** To solve this equation for p, j must be expressed in terms of p, and the solution then involves a cubic equation. For practical purposes, however, it is near enough to substitute for j the value of j_e corresponding to the economic percentage of steel; therefore,

$$p = \frac{C_s}{j_e} \qquad \textbf{(12)}$$

The error resulting from this substitution is on the safe side and for the ordinary conditions of design it need not be considered. If, however, a closer result is desired, a value of j may now be determined, that is nearly exact, by using for this purpose the value of p as determined from formula **12.** Applying formula **2,** Art. **46,** $k = \sqrt{p^2\,n^2 + 2\,p\,n} - p\,n$, and from formula **4,** Art. **47,** $j = 1 - \frac{k}{3}$. Finally, for a closer p,

$$p = \frac{C_s}{j} \qquad \textbf{(13)}$$

Example.—Using the same values as before, except that $M = 800{,}000$ inch-pounds, find the area of steel required.

Solution.—In the preceding example, it was found that $M_e = 1{,}076{,}000$ in.-lb. As this is greater than M, the steel governs the design. Applying formula **8,** $C_s = \frac{800{,}000}{18 \times 27^2 \times 12{,}500} = .00488$. As found in preceding example, $j_e = .875$. Then, applying formula **12,** $p = \frac{C_s}{j_e} = \frac{.00488}{.875} = .0056$. Substitute this value of p in formula **2,** Art. **46,** and calculate the corresponding value of k. $k = \sqrt{.0056^2 \times 15^2 + 2 \times .0056 \times 15} - .0056 \times 15 = .334$. Then, $j = 1 - \frac{k}{3} = .889$. By formula **13,** $p = \frac{C_s}{j} = \frac{.00488}{.889} = .0055$.

The area of steel $A = .0055 \times 18 \times 27 = 2.67$ sq. in. This is the same result as that previously obtained by means of Table I.

EXAMPLES FOR PRACTICE

1. In a certain beam $b=10$ inches, $d=20$ inches, $M=300{,}000$ inch-pounds, and it is decided to make $n=15$. If F_c is taken at 550 pounds per square inch and F_s at 16,000 pounds per square inch, find A.

Ans. $A=1.04$ sq. in.

2. In a certain beam $b=15$ inches, $d=30$ inches, $M=800{,}000$ inch-pounds, and it is decided to make $n=10$. If F_c is taken at 500 and F_s at 16,000, find A by use of Table I. Ans. $A=2.07$ sq. in.

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56. To review a Beam.—To review a beam is to investigate one that is already built, either for the purpose of determining its safe resisting moment for assumed values of n, F_s, and F_c or to find the stresses induced under a given load. Hence, the following two cases:

Case I.—Given, b, d, p, n, F_s, and F_c; required, M.

This problem is similar to Case II of Art. **55** and is solved by the same formulas but in this case M_s and M_c, respectively, are the unknown quantities instead of b or d, as in Art. **55.** Therefore, when p is less than p_e, the safe moment of resistance is $M_s=C_s\,b\,d^2F_s$, and when p is greater than p_e the formula $M_c=C_c\,b\,d^2F_c$ must be applied.

EXAMPLE.—It is required to find the bending moment M that can safely be resisted by a beam for which $b=15$ inches, $d=28$ inches, $p=.0072$, $n=20$, $F_s=12{,}500$, and $F_c=450$.

SOLUTION.—In this case, $r_e=\dfrac{12{,}500}{450}=27.78$. From Table I for $n=20$, or by calculation, using formula **5**, Art. **51**, p_e is found to be .0076. The percentage of steel used is less then the economic percentage, and formula **1**, Art. **53**, is applied, as the safe resisting moment of the steel must be considered. Therefore,

$$M_s=C_s\,b\,d^2\,F_s=.00621\times15\times28^2\times12{,}500=912{,}900 \text{ in.-lb.}$$

The value of C_s was taken from Table I for $n=20$, opposite $p=.0072$.

Case II.—In this case b, d, p, n, and M are known, and the actual stresses produced are to be determined. This is done by applying formulas **3** and **4**, Art. **48**, whence

$$f_c=\frac{M}{C_c\,b\,d^2} \text{ and } f_s=\frac{M}{C_s\,b\,d^2}$$

EXAMPLE.—In a certain beam, the stresses produced in the concrete and steel are to be determined. Here, $b=20$ inches, $d=36$ inches, $p=.01$, $n=18$, and $M=2,500,000$ inch-pounds.

SOLUTION.—As the value of p is given, find in Table I for $n=18$, the corresponding values of C_s and C_c. In this case, $C_s=.00851$ and $C_c=.190$. Then,

$$f_s=\frac{2,500,000}{.00851\times20\times36^2}=11,334 \text{ lb. per sq. in.;}$$

$$f_c=\frac{2,500,000}{.19\times20\times36^2}=508 \text{ lb. per sq. in.}$$

EXAMPLES FOR PRACTICE

1. For a certain beam, $b=12$ inches, $d=18$ inches, $p=.006$, $n=15$, $F_s=15,000$, and $F_c=500$. Find the resisting moment.

Ans. 295,500 in.-lb.

2. Find the stresses produced in a beam when $b=10$ inches, $d=20$ inches, $p=.0042$, $n=12$, and $M=200,000$ inch-pounds.

Ans. $\begin{cases} f_s=13,090 \text{ lb. per sq. in.} \\ f_c=407 \text{ lb. per sq. in.} \end{cases}$

57. Tables for Special Constants.—As noted before, Table I was constructed with a view of making it available for any of the variety of the constants n, F_s, and F_c ordinarily used in practice. When, however, a certain set of constants is used with great frequency a much simpler table suitable for these special constants may be prepared. In the first place, when n, F_s, and F_c are fixed, the values of the expressions C_sF_s and C_cF_c may be tabulated instead of C_s and C_c, as in Table I. The fundamental formulas then assume the form of

$$M_s=R_s\,b\,d^2 \qquad (1)$$

and

$$M_c=R_c\,b\,d^2 \qquad (2)$$

in which $R_s=C_s\,F_s$, and $R_c=C_c\,F_c$. Furthermore, in Table I both values, C_s and C_c must be given side by side with each value of p; one to be used at one time and the other at another, according as the steel ratio is below or above the economic ratio. Since this economic steel ratio is not fixed but depends for a given n on F_s and F_c, in order to determine for each par-

ticular pair of F_s and F_c which of the two coefficients C_s or C_c of Table I is to be used, the column headed r had to be provided. But when F_s and F_c are fixed the value of p_e is at once determined. Therefore, not only is the column headed r no longer needed, but instead of two columns giving for each p one coefficient for the steel and one for the concrete, it is then sufficient to tabulate the values of R_s for only the steel ratios below the economic ratio p_e and the values of R_c for only the steel ratios above the economic ratio, and place them in one continuous vertical column.

When such a column is completed, the fundamental formula expressing its practical application assumes the form of

$$M = R\, b\, d^2 \qquad (3)$$

in which M is the moment of resistance and R is the tabular value from the column above mentioned, and is equal to R_s for p less than p_e and R_c for p above p_e.

Table II contains the values of R for three sets of constants. The second and sixth columns give these values for $n=15$, $F_s=16{,}000$, and $F_c=650$. These constants are recommended by the Joint Committee for a certain grade of concrete and will be spoken of later. The third and seventh columns give the values of R for $n=12$, $F_s= 16{,}000$, and $F_c= 500$. The fourth and eighth columns give the values of R for the constants $n=15$, $F_s=16{,}000$, and $F_c=500$. The latter two sets of constants have been specified at various times by ordinances of some large cities, and have been extensively used in practice. For each of these three sets of constants the value of p_e and its corresponding R are given in Italics. For instance, for the units recommended by the Joint Committee, $p_e=.00769$ and the corresponding $R=107.53$. These figures are given in Italics in the first and second columns of the table. The values given in the column for R above the Italics were obtained from the expression $C_s F_s$, in this case by multiplying the values of C_s for $n=15$ by 16,000. Likewise, the values of R below the Italics were computed from the expression $C_c F_c$, in this case by multiplying the values of C_c for $n=15$ by 650. It will be seen that with the aid of Table I a similar table

may be prepared with but little effort for any other set of unit stresses.

58. The application of Table II will best be seen from the following examples.

EXAMPLE 1.—A beam is to be designed according to the constants recommended by the Joint Committee. The effective depth is 24 inches, the bending moment 600,000 inch-pounds, and $p=.0042$. Required, the breadth b and the amount of steel.

SOLUTION.—Here $n=15$, $F_s=16{,}000$, and $F_c=650$. Therefore, take from the second column of Table II, the value of R corresponding to $p=.0042$ which is 60.54. Then apply formula **3**, $M=R\,b\,d^2$; whence, $b=\frac{M}{R\,d^2}=\frac{600{,}000}{60.54\times24^2}=17.2$ in. Then, from the formula of Art. **46**, $A=p\,b\,d=.0042\times17.2\times24=1.73$ sq. in.

EXAMPLE 2. — A rectangular beam is to be designed using the constants $n=15$; $F_s=16000$, and $F_c=500$. The economic percentage of steel is to be used, and it is required to find b and d, and the amount of steel to resist a bending moment of 350,000 inch-pounds.

SOLUTION.—For the constants given, the value of R is to be taken from the fourth column of Table II. In this case the economic steel ratio $p_e=.00499$ and $R=71.30$. Then, applying formula **3**, $M=R\,b\,d^2$, whence $b\ d^2=\frac{M}{R}=\frac{350{,}000}{71.3}=4{,}909$. Assuming $d=20$ in., $b=\frac{4{,}909}{20^2}=12.3$ in. Then, $A=p_e\,b\,d=.00499\times12.3\times20=1.23$ sq. in.

EXAMPLES FOR PRACTICE

1. A beam 22 inches in effective depth is to be designed according to the constants recommended by the Joint Committee. The bending moment is 800,000 inch-pounds, and the economic percentage of steel is to be used. Find the values of b and A.

$$\text{Ans.}\begin{cases} b=15.4 \text{ in.} \\ A=2.61 \text{ sq. in.}\end{cases}$$

2. Find the values of b and A for the same conditions, except that $p=.009$.

$$\text{Ans.}\begin{cases} b=14.6 \text{ in.} \\ A=2.89 \text{ sq. in.}\end{cases}$$

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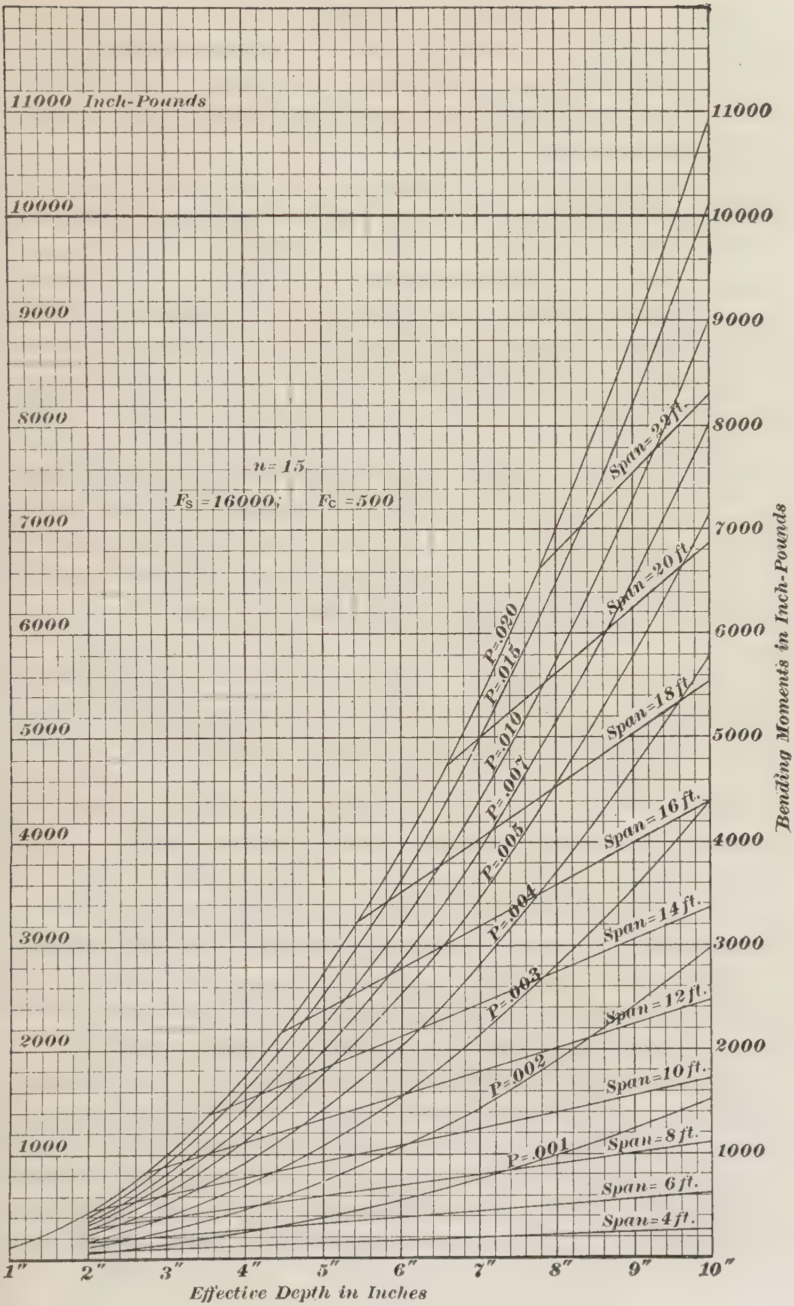
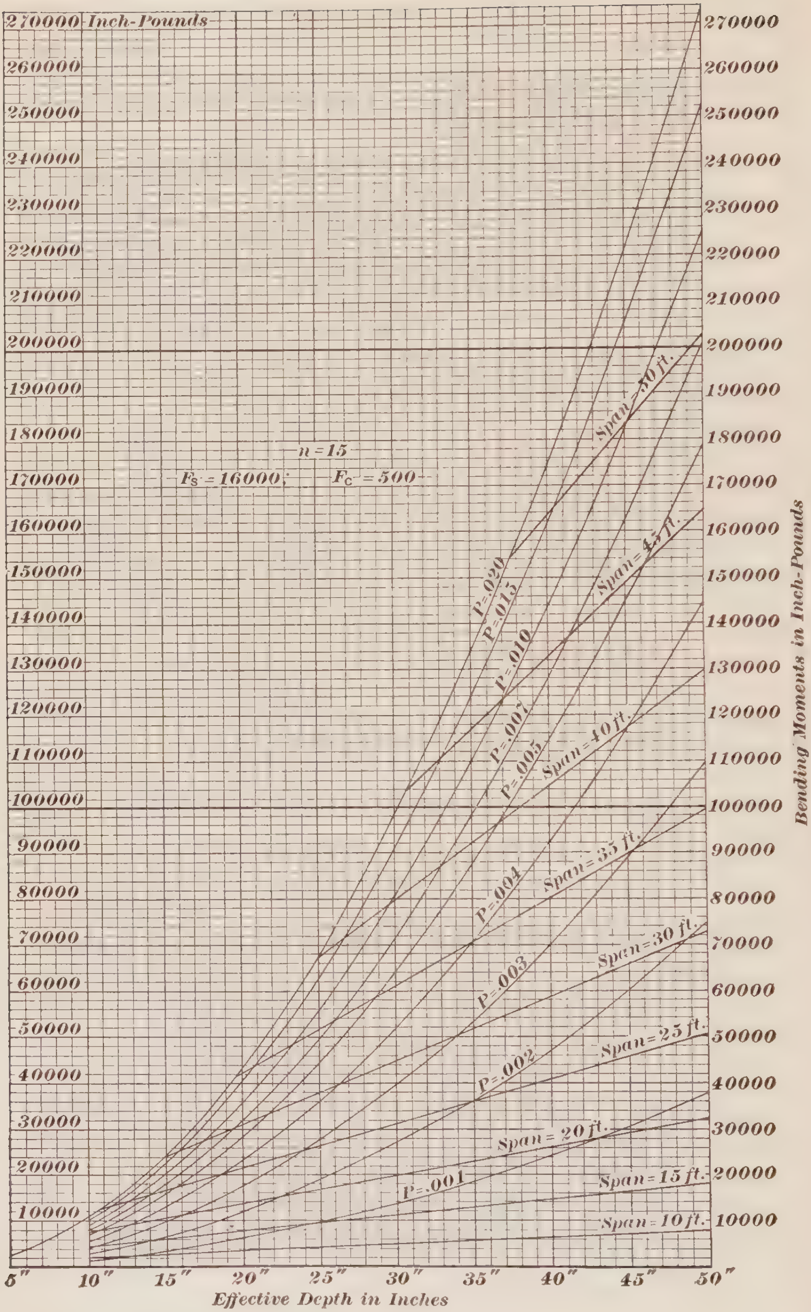
59. Weight of Beam.—In all preceding examples in designing a beam, the bending moment due to its own weight was included, for the sake of simplicity, as a part of the total


11000 Inch-Pounds
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
n=15
Fs =16000; Fc =500
P=.020
P=.015
P=.010
P=.007
P=.005
P=.004
P=.003
P=.002
P=.001
Span=22 ft.
Span=20 ft.
Span=18 ft.
Span=16 ft.
Span=14 ft.
Span=12 ft.
Span=10 ft.
Span=8 ft.
Span=6 ft.
Span=4 ft.
11000
10000
9000
8000
7000
6000
5000
4000
3000
2000
1000
Bending Moments in Inch-Pounds
1″
2″
3″
4″
5″
6″
7″
8″
9″
10″
Effective Depth in Inches

59

FIG. 47



270000 Inch-Pounds
n = 15
F_s = 16000; F_c = 500
P=.020
P=.015
P=.010
P=.007
P=.005
P=.004
P=.003
P=.002
P=.001
Span=50 ft.
Span=45 ft.
Span=40 ft.
Span=35 ft.
Span=30 ft.
Span=25 ft.
Span=20 ft.
Span=15 ft.
Span=10 ft.
Effective Depth in Inches
Bending Moments in Inch-Pounds

Fig. 48

moment M. When, however, the dimensions of the beam are to be determined, the weight cannot be known beforehand; it must, therefore, be first assumed. After the dimensions have been determined, the assumed weight can be verified by calculating its actual weight. For this purpose the weight of reinforced concrete may be taken at 150 pounds per cubic foot. If the actual weight differs materially from the one assumed, another assumption must be made and the beam redesigned. This process must be repeated if necessary until a satisfactory result is arrived at. To obviate the tedious work involved in this method the diagrams shown in Figs. 47 and 48 have been prepared. They enable one to determine approximately the dimensions of a reinforced-concrete beam that is to carry a given net moment; that is, a moment due to external loads.

In these diagrams, the abscissas represent the effective depth d in inches. In the diagram of Fig. 47 they are carried to 10 inches, and in the diagram of Fig. 48, which is drawn to a smaller scale than the one in Fig. 47, the abscissas are continued from 10 to 50 inches. The ordinates represent moments in inch-pounds; those of the curved graphs give the total resisting moment of a beam 1 inch wide when the amount of steel indicated on the curve is used; those of the straight lines give the maximum bending moment due to the weight of the same beam for the indicated spans. Consequently, the respective differences between the two sets of ordinates are the net bending moments. To facilitate the reading of the ordinates, three systems of horizontal lines were introduced, light, heavy, and extra-heavy lines. In the diagram of Fig. 47 the distance between two successive extra-heavy horizontal lines represents 10,000 inch-pounds. This distance is divided into ten equal parts, each representing 1,000 inch-pounds. These, again, are divided into five parts, each being equal to 200 inch-pounds. In the diagram of Fig. 48 the corresponding divisions represent, respectively, 100,000, 10,000, and 2,000 inch-pounds. For example, to find the net bending moment that a beam having an effective depth of 35 inches, a width of 1 inch, and reinforced with $1\frac{1}{2}$ per cent. steel, can carry on a 25-foot span, follow the ordinate marked *35″* to its intersection with the curve marked

$p = .015$. This ordinate measures one division of 100,000, two divisions of 10,000, and two divisions of 2,000 inch-pounds, a total of 124,000 inch-pounds. This is the total resisting moment of the beam. From this must be subtracted the maximum bending moment due to the weight of the beam. This is represented by the portion of the same ordinate to its intersection with the straight-line graph marked *Span* $= 25$ *ft.* and measures 36,000 inch-pounds. The difference between the two ordinates is $124{,}000 - 36{,}000 = 88{,}000$ inch-pounds, which is the net bending moment that the beam can carry on a 25-foot span.

When the net bending moment is given, and it is required to determine the effective depth of a beam 1 inch wide that can carry this moment on a certain span when a certain steel ratio is used, proceed as follows: On a straight edge of a sheet of paper lay off, to the scale of the diagram, the given bending moment, carefully marking both extremities. Place the paper on the diagram and, having the marked edge parallel with the ordinates of the diagram, shift the paper until one of the marks is on the straight-line graph for the given span and the other mark on the curve for the given steel ratio. The straight line coinciding with the edge of the paper will then be the ordinate whose abscissa represents the required effective depth. For instance, if the net bending moment is 40,000 inch-pounds, the span is 20 feet, the steel ratio is .004, and the marks at the edge of the paper where 40,000 inch-pounds had been laid off, are so situated that one mark is at the straight line marked *Span* $= 20$ *ft.* and the other at the curve marked $p = .004$, the straight line of the edge of the paper produced will intersect the abscissa at a point for which d is nearly 33 inches.

In platting the straight-line graphs for the moments due to the weight of the beam, 1 inch was added to the effective depth for Fig. 47 and 2 inches for Fig. 48. The constants used are $n = 15$, $F_s = 16{,}000$, and $F_c = 500$. It is well to note here, that when these constants are used the results are the same as those obtained when the Joint Committee units are employed, as long as p does not exceed .005. For values of p higher than .005, the constants used for the diagrams

give more conservative results, and for this reason they were selected.

The following examples will illustrate the practical application of the diagrams:

EXAMPLE 1.—A beam of 24-foot span is to carry a load of 18,000 pounds applied at the center. If the effective depth d is 30 inches, and the constants $n=12$, $F_s=16{,}000$, and $F_c=500$ are employed, find b and A, when $p=.0104$.

SOLUTION.—The net bending moment, or that due to the load, is $\frac{18{,}000\times 24}{4}\times 12=1{,}296{,}000$ in.-lb. Referring to the diagram of Fig. 48 and using the curve for $p=.010$ and the straight line for the span of 25 ft., which are the nearest graphs given in the diagram, the net bending moment for the 30-in. beam is $81{,}000-31{,}000=50{,}000$ in.-lb. per inch of width. Then the width will be $\frac{1{,}296{,}000}{50{,}000}=25.9$ in. Taking $b=26$ in., allowing 2 in. for the concrete below the reinforcement, and taking 150 lb. per cu. ft. as the weight of reinforced concrete, the weight of the beam is $\frac{26\times 32}{144}\times 150=867$ lb. per ft. The total bending moment is then $1{,}296{,}000+\frac{867\times 24\times 24}{8}\times 12=2{,}045{,}088$ in.-lb..

From Table II, for the constants given and opposite $p=.0104$, find $R=84.85$. Applying formula **3**, Art. **57**, $b=\frac{M}{R\,d^2}=\frac{2{,}045{,}088}{84.85\times 30^2}=26.7$ in. As this is greater than the assumed breadth, the weight is recalculated and the operations repeated. Taking $b=27$ in., the weight is $\frac{27\times 32}{144}\times 150=900$ lb. per ft., and the total bending moment is $1{,}296{,}000+\frac{900\times 24\times 24}{8}\times 12=2{,}073{,}600$ in.-lb. Again, applying the formula for b, $b=\frac{2{,}073{,}600}{84.85\times 30^2}=27.1$ in. As this checks closely, b may be taken as 27 in. Then, $A=.0104\times 27\times 30=8.42$ sq. in.

EXAMPLE 2.—Design a slab to carry, over a span of 10 feet, a load of 100 pounds per square foot, in addition to its own weight. The constants recommended by the Joint Committee are to be used, and $p=.0049$.

SOLUTION.—The load per inch of width and per foot of length is $\frac{100}{12}$ pounds. The maximum bending moment due to the external load is therefore $\frac{\frac{100}{12}\times 10^2}{8}\times 12=1{,}250$ in.-lb. Mark off, on the edge of a strip of

paper, 1,250 in.-lb. to the scale of Fig. 47, and move this along between the curve for $p=.005$ and the straight line for the span of 10 ft., finding the corresponding depth to be about 5.7 in. Then, the weight per foot for 1 in. width is $\frac{1\times6.7}{144}\times150=7$ lb.; and the total bending moment is 1,250 $+\frac{7\times10^2}{8}\times12=2{,}300$ in.-lb. Applying formula **3**, Art. **57**, and interpolating for R in Table II

$$d=\sqrt{\frac{M}{R\,b}}=\sqrt{\frac{2{,}300}{70.12\times1}}=5.73 \text{ in.}$$

This value agrees with the one assumed, and will be adopted. Then, $A=.0049\times1\times5.7=.0279$ sq. in. per in. of width.

EXAMPLES FOR PRACTICE

1. A beam 15 feet long is to be designed to carry a center load of 6,500 pounds. The economic percentage of steel is to be used, and $d=20$ inches. Find b and A, taking $n=15$, $F_s=16{,}000$, and $F_c=500$.

Ans. $\begin{cases} b=14 \text{ in.} \\ A=1.4 \text{ sq. in.} \end{cases}$

2. Find the effective depth of a slab and the amount of steel, the load being 200 pounds per square foot in addition to its own weight. The span is 8 feet, and the constants recommended by the Joint Committee are to be used with $p=.0038$.

Ans. $\begin{cases} d=6.5 \text{ in.} \\ A=.025 \text{ sq. in.} \end{cases}$

DOUBLE REINFORCEMENT

60. In designing a reinforced-concrete beam it sometimes occurs that the dimensions of the section are fixed, and sufficient steel cannot be provided in the bottom of the beam to carry the given load without overstressing the concrete. In such a case, to assist the resistance of the concrete in compression, steel may be placed also in the top. The beam is then said to be **double reinforced.** This method of reinforcement is not economical, but it sometimes cannot be avoided. To determine the amount of steel required to carry the given bending moment M, let M_e denote the moment that the section could resist if it were reinforced at the bottom only, with an amount of steel equal to $A_e=p_e\,b\,d$; that is, if the critical value were used. Let the line $x\,x$, Fig. 49, represent the position of the neutral axis for this arrangement. With

this reinforcement both the steel and the concrete are stressed to their maximum allowable limits F_s and F_c. Since, by the conditions of the problem, M_e balances only a part of M, let the unbalanced portion be M_x, that is, $M_x = M - M_e$. To carry this moment let the steel to be added in the bottom above the amount A_e be A_y and the amount used in the top be A_t. The simplest method of determining A_y and A_t is to proportion them so that the neutral axis shall not change its position. If this is the case, the stress f_s in the top steel may be determined by the principle given in paragraph 3, Art. **45**, namely, $\frac{f_s}{F_s} = \frac{t}{y}$, when t and y are, respectively, the distances of the top and the bottom steel from the neutral axis. Hence, $f_s = \frac{t}{y} F_s$. The total stress taken by the area A_y is then $A_y F_s$ and by the area A_t is $A_t f_s$. From the principle of mechanics, a moment, as M_x, can be balanced only by another moment of the same amount; consequently, these two forces $A_t f_s$ and $A_y F_s$ must form a couple with the lever arm q, Fig. 49, and whose moment is $M_x = A_y F_s q$; whence,

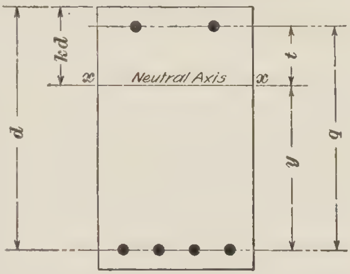


FIG. 49

$$A_y = \frac{M_x}{F_s q} \qquad (1)$$

Further, since the forces forming a couple are equal, $A_y F_s = A_t f_s$; or, substituting for f_s its value $\frac{t}{y} F_s$ above obtained and solving for A_t,

$$A_t = \frac{y}{t} A_y \qquad (2)$$

Therefore, to design a double reinforced-concrete beam, first determine A_e and M_e. Subtract the latter from M; the difference will be M_x. Then find A_y by formula **1**. The total

steel at the bottom, then, is $A = A_e + A_y$. Finally, the steel at the top is found by formula **2.**

Example.—In a certain beam b is limited to 10 inches and d to 18 inches. $M = 724{,}800$ inch-pounds, the beam is to be double reinforced, and designed for $n = 15$, $F_s = 16{,}000$, and $F_c = 500$.

Solution.—From Table II it is found that for the constants given, $p_e = .00499$ and $R = 71.3$. Then, $M_e = R\,b\,d^2 = 71.3 \times 10 \times 18^2 = 231{,}012$ in.-lb. Then, $M_x = 724{,}800 - 231{,}012 = 493{,}788$ in.-lb. If the compressive steel is placed say 2 in. from the top of the beam, then $q = d - 2 = 18 - 2 = 16$. From formula **1**, $A_y = \frac{M_x}{F_s\,q} = \frac{493{,}788}{16{,}000 \times 16} = 1.93$ sq. in. The total area of steel at the bottom is, therefore, $A = p_e\,b\,d + A_y = .00499 \times 10 \times 18 + 1.93 = 2.83$ sq. in. The area of steel at the top is found by formula **2**. As the compressive steel is 2 in. from the top of the beam, $t = k\,d - 2$, and taking the value $k = .32$ from Table I for $n = 15$ and $p = .00499$, $t = .32 \times 18 - 2 = 3.76$, and $y = d - k\,d = 18 - 5.76 = 12.24$. Then,

$$A_t = \frac{y}{t} \times A_y = \frac{12.24}{3.76} \times 1.93 = 6.28 \text{ sq. in.}$$

EXAMPLE FOR PRACTICE

In a certain beam $b = 10$ inches, $d = 24$ inches, $M = 900{,}000$ inch-pounds, and the constants recommended by the Joint Committee are to be used. The beam is to be double reinforced and the compressive steel is to be 2 inches from the top. Find A and A_t.

$$\text{Ans.} \begin{cases} A = 2.65 \text{ sq. in.} \\ A_t = 1.68 \text{ sq. in.} \end{cases}$$

CONTINUOUS BEAMS

61. When a beam rests on more than two supports it is a continuous beam. The bending moment is then reduced at the center of the span and a negative moment, inducing tension at the top and compression at the bottom of the beam, occurs at each intermediate support. Owing to the uncertainties as to the exact maximum bending moment, many engineers disregard the effect of the continuity and design the beam as if it were a simple beam. The maximum bending moment for a uniform load would then be $\frac{W\,l}{8}$, when W is the total load and l the span length. This practice should always

be followed when there is a likelihood of only one span being loaded, because in that case the loaded span acts as a simple beam. When, however, this is not the case and the supports are rigid, the Joint Committee recommends for slabs and beams having more than two spans, that for floor slabs the bending moment at the center and at the support be taken as $\frac{W l}{12}$, that for beams the bending moment at the center and at the support for interior spans be taken as $\frac{W l}{12}$, and that for end spans it be taken as $\frac{W l}{10}$ for the center and the adjoining support. Whether or not, in determining the bending moment, the continuity is taken into consideration, the top of the beam should be reinforced sufficiently with steel over each support to take the tensile stresses.

BEAMS OF T-SHAPED CROSS-SECTION

62. The great majority of reinforced-concrete beams are rectangular in cross-section, but in many cases they are built to support floor slabs, and if the concrete in the latter is intimately connected with that in the top of the beam, the two act to a great extent as one piece. The combination is shown in Fig. 50, in which $S S$ is a part of the floor slab, and B is the beam. On account of its shape, this combination is called a **T beam.** The slab $S S$ acts as a flange for the beam B; it is reinforced, as shown, by bars or shapes perpendicular to the axis of B.

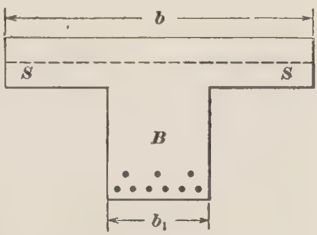


FIG. 50

When this construction is used, it is customary to design the slab first, considering it in strips 12 inches in width, and assuming that each strip acts as a separate beam. When the required depth from the top of the concrete to the steel is found, the total depth is taken from 1 to 2 inches greater, and this depth is considered to act as a flange for the beam that supports the slab.

63. The depth and the area of beam to be used in calculations depend on the manner of depositing the concrete. If the lower part of the beam is deposited in the forms and allowed to set before the slab is put on, as is frequently done in practice, there will be a weak surface where the new and the old concrete meet, and this surface cannot be relied on to transmit much stress. If, for some special reason, it is considered advisable to perform the work in this way, only the portion of the beam below the slab should be counted as effective in supporting loads, the slab being considered as simply resting on the beam. This method of construction, however, is far from economical; and it is much better practice to have, when possible, the full depth of the beam, including the slab, deposited at about the same time, or as fast as the handling of the concrete will permit. The beam and the slab then form a monolith, which, when a good quality of concrete is used, will act as one piece. But even then it is advisable to use stirrups, which should be fastened securely to the slab reinforcement. If the slab reinforcement is parallel to the beam, transverse rods should extend over the beam and into the slab a short distance on each side of the beam to distribute the compression evenly.

64. Very little is known as to the amount of slab that can be considered to act with the beam, and the present practice greatly varies in this respect. The following rules by the Joint Committee will give an idea of what the width of slab should be:

It shall not exceed one-fourth of the span length of the beam.

Its overhanging width on either side of the web shall not exceed four times the thickness of the slab.

If the neutral axis does not fall below the bottom of the slab, the beam may be designed by the method given for rectangular beams, because the parts that are missing to make it rectangular are the parts below the neutral axis. As these parts are in tension they do not materially contribute to the strength of the beam. Thus, in Fig. 51, if the neutral axis comes in the slab *a b j e*, the beam can be designed as a rectangular beam

a b c d, because the sections *e f g d* and *i j c h*, even if they were filled in, would be in tension and neglected. Whether the neutral axis is in the slab or not can be determined by formula **2**, Art. **46**, or from Table I. If it is not in the slab the formula will not give its exact location, but will indicate simply that it is not in the slab.

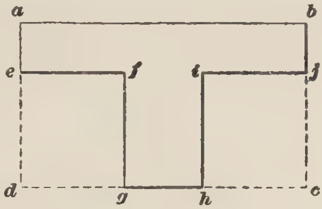


FIG. 51

65. If the neutral axis is not in the slab the following approximate method will often give results sufficiently close: Fig. 52 (*a*) shows a section of a **T** beam, together with its reinforcement. A side elevation of part of the beam is shown in (*b*). In (*a*), the neutral axis is shown below the slab at *x x*. In (*b*), the intensity of compression at different points of the beam is represented by the diagram *c m o*. If the width of stem b_1 is insignificant in comparison to the width of flange *b*, the compression caused by that part of the compression area in (*b*) represented by *e n o* may be neglected, and the entire compression may be considered as represented by the area *c m n e*. Now, since *e n* is shorter than *c m*, it is evident that the center of gravity of this area is less than $\frac{1}{2}t$ from the top of the slab. The arm of the couple between the concrete and the steel is therefore greater than $\left(d-\frac{t}{2}\right)$.

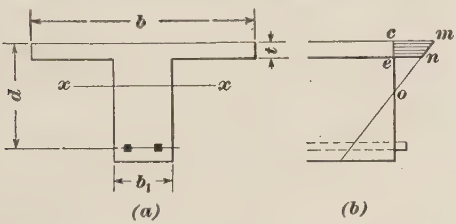


FIG. 52

It is therefore safe to assume that the resisting moment, in terms of the stress in the steel, is

$$M = A F_s \left(d-\frac{t}{2}\right), \qquad (1)$$

in which all the letters have their former significance. This formula insures sufficient steel, and is the one used for design.

To insure that the concrete is not overstressed, an expression for M in terms of F_c must be found. If F_m is the average stress in the concrete, then $M = t\,b\,F_m\left(d - \frac{t}{2}\right)$. It will be evident from an inspection of Fig. 52 *(b)*, in which $c\,m$ may represent F_c and the area $c\,m\,n\,e$ the total stress in the concrete, that F_m is less than F_c; but it is greater than $\frac{F_c}{2}$, for $F_m = \frac{c\,m + e\,n}{2} = \frac{F_c}{2} + \frac{e\,n}{2}$. Consequently, if in the above equation $\frac{F_c}{2}$ is substituted for F_m, the error will be on the safe side. The equation then becomes $M = \frac{1}{2}\,t\,b\,F_c\left(d - \frac{t}{2}\right)$, whence

$$F_c = \frac{2\,M}{t\,b\left(d - \frac{t}{2}\right)}, \qquad (2)$$

which formula may be used for finding whether the concrete is overstressed. It must not be lost sight of that in formulas **1** and **2** the compressional area of the stem is neglected. They should therefore not be used when the latter forms a considerable part of the section, which will happen when the beam is large and the slab shallow. In this case it is best to neglect the **T** effect and consider that the beam carries the whole load.

It must also be kept in mind that formulas **1** and **2** apply only when the neutral axis is below the flange.

66. As an example, a **T** beam is on a span of 16 feet. It carries a load of 5,000 pounds per foot, including its own weight. It is built solid with a slab 6 inches thick, and the total depth of the beam is 31 inches. The value of d is taken at 29 inches and the width b_1, Fig. 52, of the stem is 16 inches. Design the beam, assuming $F_s = 16{,}000$, $F_c = 500$, and $n = 15$.

The maximum bending moment is $\frac{5{,}000 \times 16 \times 16 \times 12}{8} = 1{,}920{,}000$ inch-pounds. The breadth of the flange, or b, according to the first rule in Art. **64**, must not be greater than

$\frac{1}{4} \times 16 = 4$ feet, or 48 inches. According to the second rule, b must not exceed $16+2(4\times 6)=64$ inches. As the first rule gives the smaller value for b, it will be used. To determine whether the neutral axis falls in or below the flange, the method described in Art. **55** for Case III is used. Here $C_s = \frac{1,920,000}{48\times 29^2\times 16,000} = .00297$, and $C_c = \frac{1,920,000}{48\times 29^2 \times 500} = .0951$.

A reference to Table I for $n=15$ will show that since the value of C_s requires a greater amount of steel it governs the design; the corresponding $k=.2724$, nearly; hence, $x=k\,d = .2724\times 29 = 7.9$, which would be the distance of the neutral axis from the top of the beam if the slab were rectangular. As the slab is 6 inches thick, the neutral axis falls below the flange; therefore, the method of Art. **65** will be used. Applying formula **1** of that article, and substituting known values, $1,920,000 = A\times 16,000(29-3)$, whence

$$A = 4.62 \text{ square inches}$$

It now remains to be seen whether the concrete is overstressed. This may be found by formula **2**. Substituting known values in the formula, $F_c = \frac{2\times 1,920,000}{6\times 48\,(29-3)} = 513$ pounds, which may be considered safe.

67. In designing continuous T beams it must be remembered that at each support the bending moment is reversed; therefore, the lower part of the beam is in compression and the upper part in tension. In such a case all the compression comes on the stem, and the slab, being in tension, is assumed to carry no stress at all. The beam is, in fact, a rectangular beam, and steel must then be inserted in the top to take the tension.

WORKING STRESSES AND GENERAL DETAILS

68. Value of n.—The value of n was assumed in the formulas used to be constant for different values of F_c. Strictly speaking, this is not correct, but is nearly so within the limits of the working stresses. What value of n should be selected is a matter of opinion and experience. At present, $n=12$

and $n = 15$ seem to be the values most widely used. The building laws of many cities specify what values should be used in their locality, and some of these have been mentioned in Art. **57**. The Joint Committee recommends for ordinary conditions the value $n = 15$. If the value of E_s is taken as 30,000,000, then the value of E_c, to make $n = 15$, will be 2,000,000.

69. Unit Stress in Steel.—The steel most used in reinforced concrete has a modulus of elasticity of 30,000,000 pounds per square inch, and an ultimate tensile strength of 64,000 pounds per square inch. Using a factor of safety of 4 gives 16,000 pounds per square inch for F_s. This is the unit stress recommended by the Joint Committee. It is also specified by ordinances of many large cities. On the other hand, many engineers use a much lower unit stress. The ordinances of the cities of Chicago and San Francisco specify a unit stress not exceeding one-third of the elastic limit of the material.

70. Unit Stress in Concrete.—In this Section, no definite unit stress for concrete is recommended. The stress to be used depends on how well the concrete is made, how good the materials are of which it is composed, and in what proportions these materials are mixed. Building laws often stipulate the stresses to be used. Some of these have been mentioned in Art. **57**. The Joint Committee recommends certain stresses for concrete made of Portland cement and first-class aggregates mixed in the proportion of 1 to 6; that is, 1 part of cement to 6 parts of fine and coarse aggregates. The concrete shall be of such a grade that when made under laboratory conditions of manufacture and storage, using the same consistency as used in the field, it will develop in 28 days, when cast in cylinders 8 inches in diameter and 16 inches long, a compressive stress of 2,000 pounds per square inch. For this grade of concrete the Joint Committee recommends a stress of 650 pounds per square inch for the compression in beams caused by bending. For shear proper, a unit stress of 120 pounds per square inch is recommended. The safe bond-

ing stress between concrete and plain reinforcing bars may be taken as 80 pounds per square inch of surface in contact, but in the case of drawn wire only 40 pounds per square inch is allowed.

71. Live-Load Stresses.—The preceding stresses recommended for concrete and steel are for static loads. If a live load is suddenly applied, or is moving or vibrating so that the structure is subject to impact, vibrations, or shock, lower unit stresses must be used. A good practice is to add a certain percentage to the live load and treat it as a static load.

72. Web Stresses.—One of the most frequent forms of failure of concrete beams is shown in Fig. 19. It is commonly called shear, although it is not really shear but diagonal tension that causes the cracks. The exact conditions existing in the beam causing this failure are unknown, but the methods given in this Section for preventing such failures agree with the best current practice. Their application, however, requires good judgment and practical experience. Two general methods of preventing these cracks are used, namely, by bending up part of the horizontal reinforcement and by the use of special shear members, or stirrups.

The first method is based on the following principle: At the center of the span of a uniformly loaded beam the bending moment is a maximum. Near the supports the bending moment is comparatively small, and therefore some of the steel at these places, such as the rods *b*, Fig. 10, may be bent as shown. These rods run directly across the place where the cracks occur. It is best not to bend up all the rods at one place, but to bend them in pairs at intervals, so as to distribute their effect. These bent-up rods may either be secured at the end to increase their grip, or, better, be made to extend into the adjoining beam and thus form part, at least, of the reinforcement to resist negative bending moment over the supports. Often about one-half of the main reinforcing rods may be used that way.

73. When stirrups are used for the purpose of preventing failure by diagonal tension, they serve the additional purpose

of tying the beam proper to the slab when there is one. Care must be taken to see that they cannot pull out. They are usually tied to the slab reinforcement or bent over at the ends. Deformed bars give a better grip than plain bars. The Joint Committee recommends that the distance apart of stirrups should never be more than three-quarters of the depth of the beam.

The following formulas may be employed for the purpose of designing stirrups:

Let V = total external vertical shear, in pounds;
v = unit shear, in pounds per square inch;
P = total stress in one stirrup, in pounds;
c = horizontal spacing of stirrups, in inches.

The other letters have the same meaning as previously given.

Then, for rectangular beams reinforced at the bottom,

$$v=\frac{V}{b\,j\,d} \qquad (1)$$

For vertical stirrups,

$$P=\frac{V\,c}{j\,d} \qquad (2)$$

For stirrups inclined at 45°,

$$P=0.7\,\frac{V\,c}{j\,d} \qquad (3)$$

For **T** beams,

$$v=\frac{V}{b_1\,j\,d} \qquad (4)$$

If the neutral axis is in the slab, j can be found as in rectangular beams; if it is in the stem, the formulas for rectangular beams will not give the correct value of j, and in place of $j\,d$ the approximate value of $d-\frac{t}{2}$ may be used.

74. As an example, assume that a rectangular beam is on a span of 20 feet and carries a total load of 1,500 pounds per foot. Assume that $b=12$ inches and that $d=21$ inches.

The maximum shear is equal to either reaction and is $\frac{1,500\times 20}{2}=15,000$ pounds. Assume that j has been found to be equal to $\frac{7}{8}$. Then the unit shear is $v=\frac{V}{b\,j\,d}=\frac{15,000}{12\times\frac{7}{8}\times 21}$ $=68$ pounds. The Joint Committee states that for concrete only, without any web reinforcement, the unit shear must not exceed 40 pounds per square inch; with bent-up main-reinforcement rods properly placed to resist the tendency to crack, it must not exceed 60 pounds per square inch; and with stirrups and bent-up rods it must not exceed 120. As the shear in this case is over 60, stirrups and truss rods will be used.

The size of stirrups and the allowable total stress in each one must now be decided on. For this example, let it be assumed that the allowable stress in each stirrup is 6,000 pounds and that the stirrups are vertical. It is generally assumed that the concrete itself can take one-third of the shear; therefore, the stirrups have to resist only two-thirds of the shear, or $\frac{2}{3}\times 15,000=10,000$ pounds. Substituting the correct values in formula **2**, Art. **73**, and solving for c the horizontal spacing between stirrups,

$$c=\frac{P\,j\,d}{V}=\frac{6,000\times\frac{7}{8}\times 21}{10,000}=11 \text{ inches, nearly}$$

This is the spacing near the support, where the external vertical shear is 15,000 pounds. Near the center of the beam the external vertical shear becomes less, and the spacing of the stirrups will become greater, but it should never be made greater than $\frac{3}{4}\,d$.

Another means to assist in preventing cracks due to diagonal tension, and one that is recommended by the Joint Committee, is to arrange the horizontal reinforcement so that its unit stresses will be relatively low at points of high shear.

75. Bond Between Steel and Concrete.—In a reinforced-concrete beam the stress from the load is transmitted to the steel reinforcement by means of the adhesion, or bond, between the concrete and the steel. The amount of stress H that is transmitted to the horizontal reinforcement at the

bottom at any section can be found approximately by the formula $H=\frac{V}{j\,d}$, in which V is the external shear at the section under consideration and $j\,d=D$, as before. If f_b denotes the unit bond induced at the same section and O the sum of the perimeters of the horizontal reinforcement, then O will also be the total bond area for one unit of length; therefore,

$$f_b=\frac{H}{O}=\frac{V}{j\,d\,O}$$

This value should not exceed the allowable unit adhesion between the steel and concrete. The safe unit bonding stress recommended by the Joint Committee is given in Art. **70.**

76. Details of Design.—All details of reinforced-concrete beams must be studied with extreme care, as it is on the little things that the stability of the structure sometimes depends.

77. Where, as is often the case, two reinforcing rods or bars overlap, the overlapping length must be such that the bond between the concrete and each overlapping bar shall be equal to the tension in the bar. If L is the length of lap, A the area and o the perimeter of one bar or rod, and f_b the working adhesive strength, or bond, per square inch, then $f_s\,A=f_b\,o\,L$; whence,

$$L=\frac{f_s}{f_b}\times\frac{A}{o}$$

78. Sufficient concrete must be placed beneath the reinforcing rods to protect them from fire or rust. The Joint Committee recommends 2 inches for girders, 1½ inches for beams, and 1 inch for slabs for ordinary cases. The edges of all beams and columns should be chamfered.

79. Spacing of Bars.—In placing bars, sufficient concrete should be allowed also at the sides. The Joint Committee recommends that "The lateral spacing of parallel bars should not be less than 2½ diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars should not be less than ½ inch."

COLUMNS

METHODS OF DESIGN

80. The present practice in designing reinforced-concrete columns is not at all uniform. Some engineers make the columns large enough for the concrete to carry the entire load, and put in steel rods as a precaution against any slight bending that may occur. In places where floor space is valuable, this practice, although the safest, is not economical, since the columns must be made inconveniently large, and take up space needed for other purposes. In order to save space, many engineers make the columns smaller, and reinforce them with steel in such a manner that they can safely carry the same load as larger columns not reinforced. The load is then assumed to be properly distributed between the steel and the concrete. Since the two materials must compress the same amount, the stresses in them must be in proportion to their moduli of elasticity.

81. Amount of Reinforcement.—Up to the present time, not enough experiments have been made to indicate just what percentage of reinforcement is best, and the only safe thing to do is to follow good conservative practice. From the standpoint of economy of floor space, it is advisable to use as much steel as possible, and columns have been built with high percentages of reinforcement. The smaller the cross-section of the column, the more space and concrete are saved. From the standpoint of safety, however, there is a limit that should not be exceeded. When there is too much steel in the cross-section, it is impossible to transmit to it the proper amount of stress. The result is that the concrete, being overloaded, slips on the steel rods, and may cause the collapse of the column. There is also danger, when there is a large proportion of steel, that the concrete will break off outside the rods and allow them to buckle or bend outwards.

82. Formulas for Design.—As stated in Art. **80,** the stresses in the concrete and in the steel must be in proportion

to the moduli of elasticity of the two materials. Under this condition, the shortening of the steel will be the same as that of the concrete, and there will be no tendency to sliding of one material on the other. The condition just described is stated algebraically by the proportion

$$s_s : s_c = E_s : E_c$$

whence,
$$s_s = \frac{E_s}{E_c} s_c = n\, s_c \qquad (1)$$

In this formula s_s is the unit stress in the steel, and s_c is the unit stress in the concrete.

In designing a reinforced-concrete column, only the compressive stresses are taken into account, such columns being usually not long enough to require that bending should be considered.

The units being the inch and the pound, let

a = cross-sectional area of column;
a_s = cross-sectional area of steel;
a_c = cross-sectional area of concrete;
p = ratio of steel area to total area $= \frac{a_s}{a}$;
W = total load on column, centrally applied.

Then,
$$W = a_s\, s_s + a_c\, s_c$$

But $s_s = n\, s_c$; therefore,

$$W = a_s\, n\, s_c + a_c\, s_c = s_c(a_s\, n + a_c) \qquad (2)$$

Now, $a_s = a\, p$; also, $a_c = a - a_s = a - a\, p = a(1-p)$. Substituting the values for a_s and a_c in formula **2**, it becomes

$$W = s_c\,[a\, p\, n + a(1-p)] = s_c\, a[p\, n + (1-p)]$$

Rearranging the terms,

$$W = s_c\, a\,[1 + (n-1)p] \qquad (3)$$

83. As an example, a column is 18 inches square and contains eight rods each $\frac{3}{4}$ inch square. What safe load will it carry? It is decided to limit s_s to 16,000 pounds per square inch, s_c to 450 pounds per square inch, and to make n equal to 15. The following values are then known: $a = 18 \times 18$

$=324$ square inches; $a_s=4.5$ square inches; $a_c=324-4.5=319.5$ square inches. Assume that $s_s=16{,}000$ pounds per square inch.

Then, by formula **1**, Art. **82**, $s_c=\frac{s_s}{n}=16{,}000\div15=1{,}067$ pounds per square inch. But it has been decided to keep s_c at 450. It can, therefore, be seen that the stress in the steel must be reduced; in other words, under ordinary working stresses, the steel is never compressed in columns up to its safe limit. Therefore, assume $s_c=450$ pounds per square inch. Then, $s_s=15\times450=6{,}750$. Also, from formula **2**, Art. **82**,

$$W=s_c(a_s\,n+a_c)=450\times(4.5\times15+319.5)=174{,}150 \text{ pounds.}$$

EXAMPLES FOR PRACTICE

1. A column 12 inches square has 4 square inches of steel. If s_c is taken as 400 pounds per square inch and n is taken as 12, what safe load will it carry? Ans. 75,200 lb.

2. A column 15 inches in diameter is reinforced with four $1\frac{1}{4}$-inch round bars. If a stress of 450 pounds per square inch is allowed in the concrete and if n is taken as 15, what load will it carry? Ans. 110,460 lb.

84. Hooped Columns.—The effect of hooping is to toughen the concrete. It seems to have little effect within the elastic limit, but it renders the concrete more trustworthy and thus permits higher stresses to be used. In hooped columns only the concrete within the hooping is considered in the design. The Joint Committee recommends that, when bands or hoops are used, the total amount of such reinforcement shall not be less than 1 per cent. of the volume of the column enclosed. The clear spacing of such bands or hoops shall not be greater than one-fourth of the diameter of the enclosed column.

85. Fireproofing.—If a fire occurs in a building it is probable that the surfaces of columns will be injured. If it is desired to design columns to withstand such effects, additional material must be used. The Joint Committee therefore recommends that, under ordinary conditions, the outside layer of concrete columns $1\frac{1}{2}$ inches thick be not considered

in calculating the strength of the column but be put on extra, so that in case of fire this coat may be injured and the column will still be sufficiently strong.

The Joint Committee recommends also that the steel be embedded in the concrete at least 2 inches. Attention is called to the fact that since, in hooped columns, only the concrete within the hooping is considered in calculating the ability to carry loads, and since the steel is embedded 2 inches from the surface, in reality, in hooped columns, more than $1\frac{1}{2}$ inches of fireproofing is used; whereas, in straight reinforcement, only $1\frac{1}{2}$ inches is required.

86. Length of Columns.—It is customary to neglect the effect of lateral deflection in concentrically loaded reinforced-concrete columns. Experience indicates that up to a certain ratio of the length of the column to its least dimension the strength of a column is independent of its length. But when this ratio is exceeded the flexural features no doubt play an important part. There are no formulas covering the design of columns in which this method of failure is considered, and, moreover, it is not considered safe to use long columns. Therefore, specifications usually limit the length of reinforced-concrete columns, so that they will fail by crushing or shearing and not by bending. What this limit in length should be differs in the minds of many engineers. The Joint Committee and some city ordinances recommend that the length of a reinforced column should not be greater than 15 times its least width or its diameter.

87. Stresses Employed.—The stresses generally used in practice for the design of reinforced-concrete columns depend on the specifications, the grade of concrete, city ordinances, and the experience of the engineer. The Joint Committee recommends that, for the grade of concrete mentioned in Art. **70** and for columns with longitudinal reinforcement only, s_c be taken at 450 pounds per square inch; that for columns with bands or hoops properly used, s_c be taken at 540 pounds per square inch; and that for columns reinforced with not less than 1 per cent. and not more than 4 per cent. of longitudinal

bars and with bands or hoops, s_c may be taken as high as 650 pounds per square inch. The Joint Committee also recommends that the value of n for columns may be taken at 15. These values cannot be recommended for all cases, however, but must be varied according to the experience of the designer.

SUMMARY OF NOTATION AND FORMULAS

88. For convenience of reference a summary of the notation and important formulas of this Section follows:

Notation for Rectangular Beams

A = area of steel, in tension;

b = width;

C_c = section modulus coefficient for concrete;

C_s = section modulus coefficient for steel;

d = effective depth, or distance from center of steel in tension to top of beam;

E_c = modulus of elasticity of concrete;

E_s = modulus of elasticity of steel;

F_c = allowable unit stress in concrete;

F_s = allowable unit stress in steel;

f_c = actual unit stress in concrete;

f_s = actual unit stress in steel;

j = ratio of arm of stress couple to effective depth;

k = ratio of depth of neutral surface to effective depth of beam;

M = maximum bending moment;

M_c = resisting moment of concrete;

M_s = resisting moment of steel;

$n = \frac{E_s}{E_c}$;

$p = \frac{A}{b\,d}$ = steel ratio;

p_e = economic steel ratio;

$R = C_s F_s$ or $C_c F_c$ (whichever product is smaller);

$r = \frac{F_s}{F_c}$ = stress ratio.

Formulas for Rectangular Beams

$$k=\sqrt{p^2 n^2+2 p n}-p n$$

$$k=\frac{n}{n+r} \text{ (To be used for economic steel ratio)}$$

$$k=\frac{3-\sqrt{9-24 C_c}}{2}$$

$$j=1-\frac{k}{3}$$

$$p=\frac{k}{2 r}$$

$$p=\frac{k^2}{2 n (1-k)}$$

$$p=\frac{n}{2 r (n+r)}$$

$$C_c=\frac{j k}{2}$$

$$C_s=p j$$

$$M=C_c b d^2 f_c$$

$$M=C_s b d^2 f_s$$

$$M=A j d f_s$$

$$M_c=C_c b d^2 F_c$$

$$M_s=C_s b d^2 F_s$$

$$M=R b d^2$$

$$A=p b d$$

Additional Notation for Double Reinforced Rectangular Beams

A_t = area of steel in compression;
A_y = excess of area of steel in tension;
M_e = resisting moment for economic steel ratio;
$M_x = M - M_e$;
q = distance between steel in tension and steel in compression;
t = distance from neutral surface to steel in compression;
y = distance from neutral surface to steel in tension.

FORMULAS FOR DOUBLE-REINFORCED RECTANGULAR BEAMS

$$A_y = \frac{M_x}{F_s\, q}$$

$$A_t = \frac{y}{t} A_y$$

NOTATION FOR BOND AND SHEAR OF RECTANGULAR BEAMS

A = area of one bar when two bars overlap;
c = horizontal spacing of stirrups;
f_b = working adhesive strength between steel and concrete;
L = lap of two rods that are spliced;
O = sum of perimeters of horizontal tensile reinforcement;
o = perimeter of one rod;
P = total stress in one stirrup;
V = total external vertical shear, in pounds;
v = unit shear.

FORMULAS FOR BOND AND SHEAR OF RECTANGULAR BEAMS

$$v = \frac{V}{b\, j\, d}$$

$$P = \frac{V\, c}{j\, d} \text{ (for vertical stirrups)}$$

$$P = .7 \frac{V\, c}{j\, d} \text{ (for stirrups inclined at 45°)}$$

$$f_b = \frac{V}{j\, d\, O}$$

$$L = \frac{f_s}{f_b} \times \frac{A}{o}$$

ADDITIONAL NOTATION FOR T BEAMS

b_1 = width of stem;
t = thickness of flange.

FORMULAS FOR T BEAMS

$$M = A\, F_s \left(d - \frac{t}{2}\right)$$

$$F_c = \frac{2\, M}{t\, b\left(d - \frac{t}{2}\right)}$$

$$v = \frac{V}{b_1\,j\,d}$$

Additional Notation for Columns

a = area of cross-section;
a_c = area of concrete;
a_s = area of steel;
s_c = unit stress in concrete;
s_s = unit stress in steel;
W = total load centrally applied.

Formulas for Columns

$$s_s = n\,s_c$$
$$W = s_c\,(a_s\,n + a_c)$$
$$W = s_c\,a\,[1 + (n-1)p]$$

TABLE I

PROPERTIES OF SECTIONS OF REINFORCED-CONCRETE BEAMS

$n = 8$

p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$	p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$
.0002	.0550	.9817	.0001963	.02699	137.48	.0102	.3305	.8898	.009076	.1471	16.20
.0004	.0769	.9744	.0003898	.03745	96.08	.0104	.3331	.8890	.009245	.1481	16.02
.0006	.0933	.9689	.0005813	.04520	77.75	.0106	.3357	.8881	.009414	.1491	15.83
.0008	.1069	.9644	.0007715	.05155	66.82	.0108	.3382	.8873	.009583	.1500	15.66
.0010	.1187	.9604	.0009604	.05702	59.37	.0110	.3407	.8864	.009751	.1510	15.48
.0012	.1293	.9569	.001148	.06186	53.87	.0112	.3431	.8856	.009919	.1519	15.32
.0014	.1389	.9537	.001335	.06623	49.60	.0114	.3455	.8848	.01009	.1529	15.15
.0016	.1477	.9508	.001521	.07022	46.16	.0116	.3479	.8840	.01025	.1538	15.00
.0018	.1559	.9480	.001706	.07391	43.31	.0118	.3502	.8833	.01042	.1547	14.84
.0020	.1636	.9455	.001891	.07734	40.90	.0120	.3526	.8825	.01059	.1556	14.69
.0022	.1708	.9431	.002075	.08056	38.83	.0122	.3549	.8817	.01076	.1564	14.54
.0024	.1777	.9408	.002258	.08359	37.02	.0124	.3571	.8810	.01092	.1573	14.40
.0026	.1842	.9386	.002440	.08645	35.43	.0126	.3594	.8802	.01109	.1582	14.26
.0028	.1904	.9365	.002622	.08918	34.01	.0128	.3616	.8795	.01126	.1590	14.12
.0030	.1964	.9345	.002804	.09177	32.73	.0130	.3638	.8787	.01142	.1598	13.99
.0032	.2021	.9326	.002984	.09425	31.58	.0132	.3659	.8780	.01159	.1607	13.86
.0034	.2076	.9308	.003165	.09663	30.53	.0134	.3681	.8773	.01176	.1615	13.73
.0036	.2129	.9290	.003344	.09890	29.57	.0136	.3702	.8766	.01192	.1623	13.61
.0038	.2180	.9273	.003524	.1011	28.69	.0138	.3723	.8759	.01209	.1630	13.49
.0040	.2230	.9257	.003703	.1032	27.87	.0140	.3744	.8752	.01225	.1638	13.37
.0042	.2278	.9241	.003881	.1053	27.12	.0142	.3764	.8745	.01242	.1646	13.25
.0044	.2325	.9225	.004059	.1072	26.42	.0144	.3784	.8739	.01258	.1653	13.14
.0046	.2370	.9210	.004237	.1091	25.76	.0146	.3804	.8732	.01275	.1661	13.03
.0048	.2414	.9195	.004414	.1110	25.14	.0148	.3824	.8725	.01291	.1668	12.92
.0050	.2457	.9181	.004591	.1128	24.57	.0150	.3844	.8719	.01308	.1676	12.81
.0052	.2498	.9167	.004767	.1145	24.02	.0152	.3863	.8712	.01324	.1683	12.71
.0054	.2539	.9154	.004943	.1162	23.51	.0154	.3882	.8706	.01341	.1690	12.61
.0056	.2579	.9140	.005119	.1179	23.02	.0156	.3902	.8699	.01357	.1697	12.50
.0058	.2617	.9128	.005294	.1195	22.56	.0158	.3920	.8693	.01374	.1704	12.41
.0060	.2655	.9115	.005469	.1210	22.13	.0160	.3939	.8687	.01390	.1711	12.31
.0062	.2692	.9103	.005644	.1225	21.71	.0162	.3958	.8681	.01406	.1718	12.21
.0064	.2729	.9090	.005818	.1240	21.32	.0164	.3976	.8675	.01423	.1724	12.12
.0066	.2764	.9079	.005992	.1255	20.94	.0166	.3994	.8669	.01439	.1731	12.03
.0068	.2799	.9067	.006166	.1269	20.58	.0168	.4012	.8663	.01455	.1738	11.94
.0070	.2833	.9056	.006339	.1283	20.24	.0170	.4030	.8657	.01472	.1744	11.85
.0072	.2867	.9044	.006512	.1296	19.91	.0172	.4047	.8651	.01488	.1751	11.77
.0074	.2899	.9034	.006685	.1310	19.59	.0174	.4065	.8645	.01504	.1757	11.68
.0076	.2932	.9023	.006857	.1323	19.29	.0176	.4082	.8639	.01521	.1763	11.60
.0078	.2963	.9012	.007030	.1335	19.00	.0178	.4099	.8634	.01537	.1770	11.52
.0080	.2995	.9002	.007201	.1348	18.72	.0180	.4116	.8628	.01553	.1776	11.43
.0082	.3025	.8992	.007373	.1360	18.45	.0182	.4133	.8622	.01569	.1782	11.36
.0084	.3055	.8982	.007545	.1372	18.19	.0184	.4150	.8617	.01585	.1788	11.28
.0086	.3085	.8972	.007716	.1384	17.93	.0186	.4167	.8611	.01602	.1794	11.20
.0088	.3114	.8962	.007887	.1395	17.69	.0188	.4183	.8606	.01618	.1800	11.13
.0090	.3142	.8953	.008057	.1407	17.46	.0190	.4199	.8600	.01634	.1806	11.05
.0092	.3171	.8943	.008228	.1418	17.23	.0192	.4215	.8595	.01650	.1812	10.98
.0094	.3198	.8934	.008398	.1429	17.01	.0194	.4231	.8590	.01666	.1817	10.91
.0096	.3226	.8925	.008568	.1439	16.80	.0196	.4247	.8584	.01683	.1823	10.84
.0098	.3253	.8916	.008737	.1450	16.60	.0198	.4263	.8579	.01699	.1829	10.77
.0100	.3279	8907	.008907	.1460	16.40	.0200	.4279	.8574	.01715	.1834	10.70

TABLE I—(*Continued*)

$n = 10$

p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$	p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$
.0002	.0613	.9796	.0001959	.03001	153.19	.0102	.3610	.8797	.008972	.1588	17.70
.0004	.0855	.9715	.0003886	.04155	106.92	.0104	.3638	.8787	.009139	.1598	17.49
.0006	.1037	.9654	.0005793	.05006	86.42	.0106	.3665	.8778	.009305	.1609	17.29
.0008	.1187	.9604	.0007683	.05702	74.21	.0108	.3691	.8770	.009471	.1619	17.09
.0010	.1318	.9561	.0009561	.06299	65.89	.0110	.3718	.8761	.009637	.1628	16.90
.0012	.1434	.9522	.001143	.06827	59.74	.0112	.3744	.8752	.009802	.1638	16.71
0014	.1539	.9487	.001328	.07301	54.97	.0114	.3769	.8744	.009968	.1648	16.53
0016	.1636	.9455	.001513	.07734	51.12	.0116	.3794	.8735	.010133	.1657	16.36
0018	.1726	.9425	.001696	.08133	47.94	.0118	.3819	.8727	.010298	.1667	16.18
·0020	.1810	.9397	.001879	.08504	45.25	.0120	.3844	.8719	.010462	.1676	16.02
.0022	.1889	.9370	.002061	.08851	42.93	.0122	.3868	.8711	.010627	.1685	15.85
0024	.1964	.9345	.002243	.09177	40.92	.0124	.3892	.8703	.010791	.1694	15.69
·0026	.2035	.9322	.002424	.09485	39.14	.0126	.3916	.8695	.010955	.1702	15.54
0028	.2103	.9299	.002604	.09778	37.55	.0128	.3939	.8687	.01112	.1711	15.39
0030	.2168	.9277	.002783	.1006	36.13	.0130	.3962	.8679	.01128	.1719	15.24
.0032	.2230	.9257	.002962	.1032	34.84	.0132	.3985	.8672	.01145	.1728	15.09
0034	.2290	.9237	.003140	.1057	33.67	.0134	.4007	.8664	.01161	.1736	14.95
0036	.2347	.9218	.003318	.1082	32.60	.0136	.4030	.8657	.01177	.1744	14.82
0038	.2403	.9199	.003496	.1105	31.62	.0138	.4052	.8649	.01194	.1752	14.68
0040	.2457	.9181	.003672	.1128	30.71	.0140	.4074	.8642	.01210	.1760	14.55
0042	.2509	.9164	.003849	.1149	29.86	.0142	.4095	.8635	.01226	.1768	14.42
0044	.2559	.9147	.004025	.1170	29.08	.0144	.4116	.8628	.01242	.1776	14.29
0046	.2608	.9131	.004200	.1191	28.35	.0146	.4137	.8621	.01259	.1783	14.17
0048	.2655	.9115	.004375	.1210	27.66	.0148	.4158	.8614	.01275	.1791	14.05
0050	.2702	.9099	.004550	.1229	27.02	.0150	.4179	.8607	.01291	.1798	13.93
0052	.2747	.9084	.004724	.1248	26.41	.0152	.4199	.8600	.01307	.1806	13.81
0054	.2790	.9070	.004898	.1265	25.84	.0154	.4219	.8594	.01323	.1813	13.70
0056	.2833	.9056	.005071	.1283	25.30	.0156	.4239	.8587	.01340	.1820	13.59
0058	.2875	.9042	.005244	.1300	24.78	.0158	.4259	.8580	.01356	.1827	13.48
0060	.2916	.9028	.005417	.1316	24.30	.0160	.4279	.8574	.01372	.1834	13.37
0062	.2956	.9015	.005589	.1332	23.84	.0162	.4298	.8567	.01388	.1841	13.27
0064	.2995	.9002	.005761	.1348	23.39	.0164	.4317	.8561	.01404	.1848	13.16
0066	.3033	.8989	.005933	.1363	22.97	.0166	.4336	.8555	.01420	.1855	13.06
0068	.3070	.8977	.006104	.1378	22.57	.0168	.4355	.8548	.01436	.1861	12.96
0070	.3107	.8964	.006275	.1392	22.19	.0170	.4374	.8542	.01452	.1868	12.86
0072	.3142	.8953	.006446	.1407	21.82	.0172	.4392	.8536	.01468	.1875	12.77
0074	.3178	.8941	.006616	.1421	21.47	.0174	.4410	.8530	.01484	.1881	12.67
0076	.3212	.8929	.006786	.1434	21.13	.0176	.4429	.8524	.01500	.1887	12.58
0078	.3246	.8918	.006956	.1447	20.81	.0178	.4446	.8518	.01516	.1894	12.49
0080	.3279	.8907	.007126	.1460	20.50	.0180	.4464	.8512	.01532	.1900	12.40
0082	.3312	.8896	.007295	.1473	20.19	.0182	.4482	.8506	.01548	.1906	12.31
0084	.3344	.8885	.007464	.1486	19.90	.0184	.4499	.8500	.01564	.1912	12.23
0086	.3376	.8875	.007632	.1498	19.63	.0186	.4516	.8495	.01580	.1918	12.14
0088	.3407	.8864	.007801	.1510	19.36	.0188	.4534	.8489	.01596	.1924	12.06
0090	.3437	.8854	.007969	.1522	19.09	.0190	.4551	.8483	.01612	.1930	11.98
0092	.3467	.8844	.008137	.1533	18.84	.0192	.4567	.8478	.01628	.1936	11.89
0094	.3497	.8834	.008304	.1545	18.60	.0194	.4584	.8472	.0164	.1942	11.81
0096	.3526	.8825	.008472	.1556	18.36	.0196	.4601	.8466	.0166	.1948	11.74
0098	.3554	.8815	.008639	.1567	18.13	.0198	.4617	.8461	.0168	.1953	11.66
0100	.3583	.8806	.008806	.1577	17.91	.0200	.4633	.8456	.0169	.1959	11.58

TABLE I—(*Continued*)

$n = 12$

p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$	p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$
.0002	.0669	.9777	.0001955	.0327	167.30	.0102	.3873	.8709	.008883	.1686	18.98
.0004	.0933	.9689	.0003876	.0452	116.62	.0104	.3902	.8700	.009047	.1697	18.76
.0006	.1130	.9623	.0005774	.0544	94.18	.0106	.3930	.8690	.009211	.1707	18.54
.0008	.1293	.9569	.0007655	.0619	80.81	.0108	.3958	.8681	.009375	.1718	18.32
.0010	.1434	.9522	.0009522	.0683	71.69	.0110	.3985	.8672	.009539	.1728	18.11
.0012	.1559	.9480	.001138	.0739	64.96	.0112	.4012	.8663	.009702	.1738	17.91
.0014	.1673	.9442	.001322	.0790	59.74	.0114	.4039	.8654	.009865	.1747	17.71
.0016	.1777	.9408	.001505	.0836	55.53	.0116	.4065	.8645	.010028	.1757	17.52
.0018	.1874	.9375	.001688	.0878	52.05	.0118	.4091	.8636	.010191	.1766	17.33
.0020	.1964	.9345	.001869	.0918	49.10	.0120	.4116	.8628	.010353	.1776	17.15
.0022	.2049	.9317	.002050	.0955	46.57	.0122	.4142	.8619	.010516	.1785	16.97
.0024	.2129	.9290	.002230	.0989	44.36	.0124	.4167	.8611	.010678	.1794	16.80
.0026	.2205	.9265	.002409	.1022	42.41	.0126	.4191	.8603	.010840	.1803	16,63
.0028	.2278	.9241	.002587	.1053	40.68	.0128	.4215	.8595	.01100	.1812	16.47
.0030	.2347	.9218	.002765	.1082	39.12	.0130	.4239	.8587	.01116	.1820	16.31
.0032	.2414	.9195	.002943	.1110	37.72	.0132	.4263	.8579	.01132	.1829	16.15
.0034	.2478	.9174	.003119	.1136	36.43	.0134	.4287	.8571	.01149	.1837	15.99
.0036	.2539	.9154	.003295	.1162	35.26	.0136	.4310	.8563	.01165	.1845	15.84
.0038	.2598	.9134	.003471	.1187	34.19	.0138	.4333	.8556	.01181	.1853	15.70
.0040	.2655	.9115	.003646	.1210	33.19	.0140	.4355	.8548	.01197	.1861	15.55
.0042	.2711	.9096	.003821	.1233	32.27	.0142	.4377	.8541	.01213	.1869	15.41
.0044	.2764	.9079	.003995	.1255	31.41	.0144	.4399	.8534	.01229	.1877	15.28
.0046	.2816	.9061	.004168	.1276	30.61	.0146	.4421	.8526	.01245	.1885	15.14
.0048	.2867	.9044	.004341	.1296	29.86	.0148	.4443	.8519	.01261	.1892	15.01
.0050	.2916	.9028	.004514	.1316	29.16	.0150	.4464	.8512	.01277	.1900	14.88
.0052	.2963	.9012	.004686	.1335	28.49	.0152	.4485	.8505	.01293	.1907	14.75
.0054	.3010	.8997	.004858	.1354	27.87	.0154	.4506	.8498	.01309	.1915	14.63
.0056	.3055	.8982	.005030	.1372	27.28	.0156	.4527	.8491	.01325	.1922	14.51
.0058	.3099	.8967	.005201	.1390	26.72	.0158	.4547	.8484	.01341	.1929	14.39
.0060	.3142	.8953	.005372	.1407	26.19	.0160	.4567	.8478	.01356	.1936	14.27
.0062	.3185	.8938	.005542	.1423	25.68	.0162	.4587	.8471	.01372	.1943	14.16
.0064	.3226	.8925	.005712	.1439	25.20	.0164	.4607	.8464	.01388	.1950	14.05
.0066	.3266	.8911	.005881	.1455	24.74	.0166	.4627	.8458	.01404	.1957	13.94
.0068	.3305	.8898	.006051	.1471	24.30	.0168	.4646	.8451	.01420	.1963	13.83
.0070	.3344	.8885	.006220	.1486	23.89	.0170	.4665	.8445	.01436	.1970	13.72
.0072	.3382	.8873	.006388	.1500	23.48	.0172	.4684	.8439	.01451	.1976	13.62
.0074	.3419	.8860	.006557	.1515	23.10	.0174	.4703	.8432	.01467	.1983	13.51
.0076	.3455	.8848	.006725	.1529	22.73	.0176	.4722	.8426	.01483	.1989	13.41
.0078	.3491	.8836	.006892	.1542	22.38	.0178	.4740	.8420	.01499	.1996	13.32
.0080	.3526	.8825	.007060	.1556	22.04	.0180	.4758	.8414	.01514	.2002	13.22
.0082	.3560	.8813	.007227	.1569	21.71	.0182	.4777	.8408	.01530	.2008	13.12
.0084	.3594	.8802	.007394	.1582	21.39	.0184	.4795	.8402	.01546	.2014	13.03
.0086	.3627	.8791	.007560	.1594	21.09	.0186	.4812	.8396	.01562	.2020	12.94
.0088	.3659	.8780	.007727	.1607	20.79	.0188	.4830	.8390	.01577	.2026	12.85
.0090	.3691	.8770	.007893	.1619	20.51	.0190	.4847	.8384	.01593	.2032	12.76
.0092	.3723	.8759	.008058	.1630	20.23	.0192	.4865	.8378	.01609	.2038	12.67
.0094	.3754	.8749	.008224	.1642	19.97	.0194	.4882	.8373	.01624	.2044	12.58
.0096	.3784	.8739	.008389	.1653	19.71	.0196	.4899	.8367	.01640	.2049	12.50
.0098	.3814	.8729	.008554	.1665	19.46	.0198	.4915	.8362	.01656	.2055	12.41
.0100	.3844	.8719	.008719	.1676	19.22	.0200	.4932	.8356	.01671	.2061	12.33

TABLE I—(*Continued*)

$n = 15$

p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$	p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$
.0002	.0745	.9752	.0001950	.0363	186.30	.0102	.4209	.8597	.008769	.1809	20.63
.0004	.1037	.9654	.0003862	.0501	129.64	.0104	.4239	.8587	.008930	.1820	20.38
.0006	.1255	.9582	.0005749	.0601	104.56	.0106	.4269	.8577	.009092	.1831	20.14
.0008	.1434	.9522	.0007618	.0683	89.62	.0108	.4298	.8567	.009253	.1841	19.90
.0010	.1589	.9470	.0009470	.0752	79.43	.0110	.4327	.8558	.009413	.1851	19.67
.0012	.1726	.9425	.001131	.0813	71.92	.0112	.4355	.8548	.009574	.1861	19.44
.0014	.1850	.9383	.001314	.0868	66.08	.0114	.4383	.8539	.009734	.1871	19.22
.0016	.1964	.9345	.001495	.0918	61.38	.0116	.4410	.8530	.009895	.1881	19.01
.0018	.2069	.9310	.001676	.0963	57.48	.0118	.4438	.8521	.010055	.1891	18.80
.0020	.2168	.9277	.001855	.1006	54.20	.0120	.4464	.8512	.010214	.1900	18.60
.0022	.2260	.9247	.002034	.1045	51.37	.0122	.4491	.8503	.010374	.1909	18.40
.0024	.2347	.9218	.002212	.1082	48.90	.0124	.4516	.8495	.010533	.1918	18.21
.0026	.2430	.9190	.002389	.1117	46.73	.0126	.4542	.8486	.010692	.1927	18.02
.0028	.2509	.9164	.002566	.1149	44.80	.0128	.4567	.8478	.010851	.1936	17.84
.0030	.2584	.9139	.002742	.1181	43.06	.0130	.4592	.8469	.01101	1945	17.66
.0032	.2655	.9115	.002917	.1210	41.49	.0132	.4617	.8461	.01117	.1953	17.49
.0034	.2724	.9092	.003091	.1238	40.06	.0134	.4641	.8453	.01133	.1962	17.32
.0036	.2790	.9070	.003265	.1265	38.76	.0136	.4665	.8445	.01149	.1970	17.15
.0038	.2854	.9049	.003438	.1291	37.55	.0138	.4689	.8437	.01164	.1978	16.99
.0040	.2916	.9028	.003611	.1316	36.45	.0140	.4712	.8429	.01180	.1986	16.83
.0042	.2975	.9008	.003783	.1340	35.42	.0142	.4736	.8421	.01196	.1994	16.67
.0044	.3033	.8989	.003955	.1363	34.46	.0144	.4759	.8414	.01212	.2002	16.52
.0046	.3088	.8971	.004126	.1385	33.57	.0146	.4781	.8406	.01227	.2010	16.37
.0048	.3142	.8953	.004297	.1407	32.73	.0148	.4803	.8399	.01243	.2017	16.23
.0050	.3195	.8935	.004468	.1427	31.95	.0150	.4825	.8392	.01259	.2025	16.08
.0052	.3246	.8918	.004637	.1447	31.21	.0152	.4847	.8384	.01274	.2032	15.95
.0054	.3296	.8901	.004807	.1467	30.52	.0154	.4869	.8377	.01290	.2039	15.81
.0056	.3344	.8885	.004976	.1486	29.86	.0156	.4890	.8370	.01306	.2047	15.67
.0058	.3391	.8870	.005144	.1504	29.23	.0158	.4911	.8363	.01321	.2054	15.54
.0060	.3437	.8854	.005313	.1522	28.64	.0160	.4932	.8356	.01337	.2061	15.41
.0062	.3482	.8839	.005480	.1539	28.08	.0162	.4953	.8349	.01353	.2068	15.29
.0064	.3526	.8825	.005648	.1556	27.54	.0164	.4973	.8342	.01368	.2074	15.16
.0066	.3569	.8810	.005815	.1572	27.03	.0166	.4993	.8336	.01384	.2081	15.04
.0068	.3610	.8797	.005982	.1588	26.55	.0168	.5013	.8329	.01399	.2088	14.92
.0070	.3651	.8783	.006148	.1603	26.08	.0170	.5033	.8322	.01415	.2094	14.80
.0072	.3691	.8770	.006314	.1619	25.63	.0172	.5053	.8316	.01430	.2101	14.69
.0074	.3731	.8756	.006480	.1633	25.21	.0174	.5072	.8309	.01446	.2107	14.57
.0076	.3769	.8744	.006645	.1648	24.80	.0176	.5091	.8303	.01461	.2114	14.46
.0078	.3807	.8731	.006810	.1662	24.40	.0178	.5110	.8297	.01477	.2120	14.35
.0080	.3844	.8719	.006975	.1676	24.02	.0180	.5129	.8290	.01492	.2126	14.25
.0082	.3880	.8707	.007139	.1689	23.66	.0182	.5147	.8284	.01508	.2132	14.14
.0084	.3916	.8695	.007304	.1702	23.31	.0184	.5166	.8278	.01523	.2138	14.04
.0086	.3951	.8683	.007467	.1715	22.97	.0186	.5184	.8272	.01539	.2144	13.94
.0088	.3985	.8672	.007631	.1728	22.64	.0188	5202	.8266	.01554	.2150	13.84
.0090	.4019	.8660	.007794	.1740	22.33	.0190	.5220	.8260	.01569	.2156	13.74
.0092	.4052	.8649	.007957	.1752	22.02	.0192	.5238	.8254	.01585	.2162	13.64
.0094	.4084	.8639	.008120	.1764	21.73	.0194	.5255	.8248	.01600	.2167	13.54
.0096	.4116	.8628	.008283	.1776	21.44	.0196	.5272	.8243	.01616	.2173	13.45
.0098	.4148	.8617	.008445	.1787	21.16	.0198	.5290	.8237	.01631	.2178	13.36
.0100	.4179	.8607	.008607	.1798	20.89	.0200	.5307	.8231	.01646	.2184	13.27

TABLE I—(*Continued*)

$n = 18$

p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$	p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$
.0002	.0813	.9729	.0001946	.03956	203.33	.0102	.4496	.8501	.008671	.1911	22.04
.0004	.1130	.9623	.0003849	.05438	141.27	.0104	.4527	.8491	.008831	.1922	21.76
.0006	.1366	.9545	.0005727	.06517	113.81	.0106	.4557	.8481	.008990	.1933	21.50
.0008	.1559	.9480	.0007584	.07391	97.45	.0108	.4587	.8471	.009150	.1943	21.24
.0010	.1726	.9425	.0009425	.08133	86.28	.0110	.4617	.8461	.009307	.1953	20.99
.0012	.1874	.9375	.001125	.08783	78.07	.0112	.4646	.8451	.009465	.1963	20.74
.0014	.2007	.9331	.001306	.09364	71.68	.0114	.4675	.8442	.009624	.1973	20.50
.0016	.2129	.9290	.001486	.09891	66.54	.0116	.4703	.8432	.009781	.1983	20.27
.0018	.2242	.9253	.001665	.1037	62.28	.0118	.4731	.8423	.009939	.1992	20.05
.0020	.2347	.9218	.001844	.1082	58.68	.0120	.4759	.8414	.010097	.2002	19.83
.0022	.2446	.9185	.002021	.1123	55.59	.0122	.4786	.8405	.010254	.2011	19.61
.0024	.2539	.9154	.002197	.1162	52.89	.0124	.4812	.8396	.010411	.2020	19.40
.0026	.2627	.9124	.002372	.1198	50.52	.0126	.4839	.8387	.010568	.2029	19.20
.0028	.2711	.9096	.002547	.1233	48.40	.0128	.4865	.8378	.010724	.2038	19.00
.0030	.2790	.9070	.002721	.1265	46.51	.0130	.4890	.8370	.010881	.2047	18.81
.0032	.2867	.9044	.002894	.1296	44.79	.0132	.4915	.8362	.01104	.2055	18.62
.0034	.2940	.9020	.003067	.1326	43.23	.0134	.4940	.8353	.01119	.2063	18.43
.0036	.3010	.8997	.003239	.1354	41.80	.0136	.4965	.8345	.01135	.2072	18.25
.0038	.3077	.8974	.003410	.1381	40.49	.0138	.4989	.8337	.01150	.2080	18.08
.0040	.3142	.8953	.003581	.1407	39.28	.0140	.5013	.8329	.01166	.2088	17.90
.0042	.3205	.8932	.003751	.1431	38.16	.0142	.5037	.8321	.01182	.2096	17.74
.0044	.3266	.8911	.003921	.1455	37.11	.0144	.5060	.8313	.01197	.2103	17.57
.0046	.3325	.8892	.004090	.1478	36.14	.0146	.5083	.8306	.01213	.2111	17.41
.0048	.3382	.8873	.004259	.1500	35.23	.0148	.5106	.8298	.01228	.2119	17.25
.0050	.3437	.8854	.004427	.1522	34.37	.0150	.5129	.8290	.01244	.2126	17.10
.0052	.3491	.8836	.004595	.1542	33.57	.0152	.5151	.8283	.01259	.2133	16.94
.0054	.3543	.8819	.004762	.1563	32.81	.0154	.5173	.8276	.01274	.2141	16.80
.0056	.3594	.8802	.004929	.1582	32.09	.0156	.5195	.8268	.01290	.2148	16.65
.0058	.3643	.8786	.005096	.1600	31.41	.0158	.5216	.8261	.01305	.2155	16.51
.0060	.3691	.8770	.005262	.1619	30.76	.0160	.5238	.8254	.01321	.2162	16.37
.0062	.3738	.8754	.005427	.1636	30.15	.0162	.5259	.8247	.01336	.2168	16.23
.0064	.3784	.8739	.005593	.1653	29.56	.0164	.5279	.8240	.01351	.2175	16.10
.0066	.3829	.8724	.005758	.1670	29.01	.0166	.5300	.8233	.01367	.2182	15.96
.0068	.3873	.8709	.005922	.1686	28.48	.0168	.5320	.8227	.01382	.2188	15.83
.0070	.3916	.8695	.006086	.1702	27.97	.0170	.5340	.8220	.01397	.2195	15.71
.0072	.3958	.8681	.006250	.1718	27.48	.0172	.5360	.8213	.01413	.2201	15.58
.0074	.3999	.8667	.006414	.1733	27.02	.0174	.5380	.8207	.01428	.2208	15.46
.0076	.4039	.8654	.006577	.1747	26.57	.0176	.5399	.8200	.01443	.2214	15.34
.0078	.4078	.8641	.006740	.1762	26.14	.0178	.5418	.8194	.01459	.2220	15.22
.0080	.4116	.8628	.006902	.1776	25.73	.0180	.5437	.8188	.01474	.2226	15.10
.0082	.4154	.8615	.007065	.1789	25.33	.0182	.5456	.8181	.01489	.2232	14.99
.0084	.4191	.8603	.007226	.1803	24.95	.0184	.5475	.8175	.01504	.2238	14.88
.0086	.4227	.8591	.007388	.1816	24.58	.0186	.5493	.8169	.01519	.2244	14.77
.0088	.4263	.8579	.007549	.1829	24.22	.0188	.5512	.8163	.01535	.2250	14.66
.0090	.4298	.8567	.007711	.1841	23.88	.0190	.5530	.8157	.01550	.2255	14.55
.0092	.4333	.8556	.007871	.1853	23.55	.0192	.5548	.8151	.01565	.2261	14.45
.0094	.4366	.8545	.008032	.1865	23.22	.0194	.5565	.8145	.01580	.2267	14.34
.0096	.4399	.8534	008192	.1877	22.91	.0196	.5583	.8139	.01595	.2272	14.24
.0098	.4432	.8523	.008352	.1889	22.61	.0198	.5600	.8133	.01610	.2277	14.14
.0100	.4464	.8512	.008512	.1900	22.32	.0200	.5617	.8128	.01626	.2283	14.04

TABLE I—(*Continued*)

$n = 20$

p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$	p	k	j	C_s	C_c	$r=\frac{f_s}{f_c}$
.0002	.0855	.9715	.0001943	.04155	213.83	.0102	.4665	.8445	.008614	.1970	22.87
.0004	.1187	.9604	.0003842	.05702	148.43	.0104	.4697	.8434	.008772	.1981	22.58
.0006	.1434	.9522	.0005713	.06827	119.49	.0106	.4728	.8424	.008929	.1991	22.30
.0008	.1636	.9455	.0007564	.07734	102.25	.0108	.4759	.8414	.009087	.2002	22.03
.0010	.1810	.9397	.0009397	.08504	90.50	.0110	.4789	.8404	.009244	.2012	21.77
.0012	.1964	.9345	.001121	.09177	81.83	.0112	.4818	.8394	.009401	.2022	21.51
.0014	.2103	.9299	.001302	.09778	75.11	.0114	.4847	.8384	.009558	.2032	21.26
.0016	.2230	.9257	.001481	.1032	69.69	.0116	.4876	.8375	.009715	.2042	21.02
.0018	.2347	.9218	.001659	.1082	65.20	.0118	.4904	.8365	.009871	.2051	20.78
.0020	.2457	.9181	.001836	.1128	61.41	.0120	.4932	.8356	.01003	.2061	20.55
.0022	.2559	.9147	.002012	.1170	58.16	.0122	.4960	.8347	.01018	.2070	20.33
.0024	.2655	.9115	.002188	.1210	55.32	.0124	.4987	.8338	.01034	.2079	20.11
.0026	.2747	.9084	.002362	.1248	52.82	.0126	.5013	.8329	.01049	.2088	19.89
.0028	.2833	.9056	.002536	.1283	50.59	.0128	.5040	.8320	.01065	.2097	19.69
.0030	.2916	.9028	.002708	.1316	48.59	.0130	.5066	.8311	.01080	.2105	19.48
.0032	.2995	.9002	.002881	.1348	46.79	.0132	.5091	.8303	.01096	.2114	19.28
.0034	.3070	.8977	.003052	.1378	45.15	.0134	.5116	.8295	.01111	.2122	19.09
.0036	.3142	.8953	.003223	.1407	43.65	.0136	.5141	.8286	.01127	.2130	18.91
.0038	.3212	.8929	.003393	.1434	42.26	.0138	.5166	.8278	.01142	.2138	18.72
.0040	.3279	.8907	.003563	.1460	40.99	.0140	.5190	.8270	.01158	.2146	18.54
.0042	.3344	.8885	.003732	.1486	39.81	.0142	.5214	.8262	.01173	.2154	18.36
.0044	.3407	.8864	.003900	.1510	38.71	.0144	.5238	.8254	.01189	.2162	18.19
.0046	.3467	.8844	.004068	.1533	37.69	.0146	.5261	.8246	.01204	.2169	18.02
.0048	.3526	.8825	.004236	.1556	36.73	.0148	.5284	.8239	.01219	.2177	17.85
.0050	.3583	.8806	.004403	.1577	35.83	.0150	.5307	.8231	.01235	.2184	17.69
.0052	.3638	.8787	.004569	.1598	34.98	.0152	.5329	.8224	.01250	.2191	17.53
.0054	.3691	.8770	.004736	.1619	34.18	.0154	.5351	.8216	.01265	.2198	17.37
.0056	.3744	.8752	.004901	.1638	33.42	.0156	.5373	.8209	.01281	.2205	17.22
.0058	.3794	.8735	.005066	.1657	32.71	.0158	.5395	.8202	.01296	.2212	17.07
.0060	.3844	.8719	.005231	.1676	32.03	.0160	.5416	.8195	.01311	.2219	16.93
.0062	.3892	.8703	.005396	.1694	31.39	.0162	.5437	.8188	.01326	.2226	16.78
.0064	.3939	.8687	.005560	.1711	30.77	.0164	.5458	.8181	.01342	.2233	16.64
.0066	.3985	.8672	.005723	.1728	30.19	.0166	.5479	.8174	.01357	.2239	16.50
.0068	.4030	.8657	.005887	.1744	29.63	.0168	.5499	.8167	.01372	.2246	16.37
.0070	.4074	.8642	.006049	.1760	29.10	.0170	.5520	.8160	.01387	.2252	16.23
.0072	.4116	.8628	.006212	.1776	28.59	.0172	.5540	.8153	.01402	.2258	16.10
.0074	.4158	.8614	.006374	.1791	28.10	.0174	.5559	.8147	.01418	.2265	15.98
.0076	.4199	.8600	.006536	.1806	27.63	.0176	.5579	.8140	.01433	.2271	15.85
.0078	.4239	.8587	.006698	.1820	27.18	.0178	.5598	.8134	.01448	.2277	15.73
.0080	.4279	.8574	.006859	.1834	26.74	.0180	.5617	.8128	.01463	.2283	15.60
.0082	.4317	.8561	.007020	.1848	26.33	.0182	.5636	.8121	.01478	.2289	15.48
.0084	.4355	.8548	.007181	.1861	25.92	.0184	.5655	.8115	.01493	.2295	15.37
.0086	.4392	.8536	.007341	.1875	25.54	.0186	.5674	.8109	.01508	.2300	15.25
.0088	.4429	.8524	.007501	.1887	25.16	.0188	.5692	.8103	.01523	.2306	15.14
.0090	.4464	.8512	.007661	.1900	24.80	.0190	.5710	.8097	.01538	.2312	15.03
.0092	.4499	.8500	.007820	.1912	24.45	.0192	.5728	.8091	.01553	.2317	14.92
.0094	.4534	.8489	.007979	.1924	24.12	.0194	.5746	.8085	.01568	.2323	14.81
.0096	.4567	.8478	.008138	.1936	23.79	.0196	.5763	.8079	.01583	.2328	14.70
.0098	.4601	.8466	.008297	.1948	23.47	.0198	.5781	.8073	.01598	.2333	14.60
.0100	.4633	.8456	.008456	.1959	23.17	.0200	.5798	.8067	.01613	.2339	14.49

TABLE II

TABLE FOR SPECIAL CONSTANTS

p	R			p	R		
	$n=15$ $F_s=16,000$ $F_c=650$	$n=12$ $F_s=16,000$ $F_c=500$	$n=15$ $F_s=16,000$ $F_c=500$		$n=15$ $F_s=16,000$ $F_c=650$	$n=12$ $F_s=16,000$ $F_c=500$	$n=15$ $F_s=16,000$ $F_c=500$
.0002	3.12	3.13	3.12	.0102	117.61	84.32	90.47
.0004	6.18	6.20	6.18	.0104	118.31	84.85	91.01
.0006	9.20	9.24	9.20	.0106	119.00	85.37	91.54
.0008	12.19	12.25	12.19	.0108	119.68	85.89	92.06
.0010	15.15	15.24	15.15	.0110	120.34	86.39	92.57
.0012	18.10	18.20	18.10	.0112	120.99	86.89	93.07
.0014	21.02	21.15	21.02	.0114	121.64	87.37	93.57
.0016	23.92	24.08	23.92	.0116	122.27	87.85	94.05
.0018	26.81	27.00	26.81	.0118	122.89	88.32	94.53
.0020	29.69	29.91	29.69	.0120	123.50	88.79	95.00
.0022	32.55	32.80	32.55	.0122	124.10	89.25	95.46
.0024	35.40	35.67	35.40	.0124	124.69	89.70	95.91
.0026	38.23	38.54	38.23	.0126	125.27	90.14	96.36
.0028	41.05	41.40	41.05	.0128	125.84	90.58	96.80
.0030	43.87	44.24	43.87	.0130	126.40	91.01	97.23
.0032	46.67	47.08	46.67	.0132	126.96	91.43	97.66
.0034	49.46	49.91	49.46	.0134	127.51	91.85	98.08
.0036	52.24	52.73	52.24	.0136	128.04	92.26	98.50
.0038	55.02	55.53	55.02	.0138	128.57	92.67	98.90
.0040	57.78	58.34	57.78	.0140	129.10	93.07	99.31
.0042	60.54	61.13	60.54	.0142	129.61	93.47	99.70
.00426		*61.98*					
.0044	63.28	62.74	63.28	.0144	130.12	93.86	100.09
.0046	66.02	63.80	66.02	.0146	130.62	94.24	100.48
.0048	68.76	64.82	68.76	.0148	131.12	94.62	100.86
.00499			*71.30*				
.0050	71.48	65.81	71.37	.0150	131.60	95.00	101.23
.0052	74.20	66.77	72.37	.0152	132.08	95.37	101.60
.0054	76.91	67.70	73.34	.0154	132.56	95.73	101.97
.0056	79.61	68.60	74.28	.0156	133.02	96.09	102.33
.0058	82.31	69.48	75.19	.0158	133.49	96.45	102.68
.0060	85.00	70.33	76.08	.0160	133.94	96.80	103.03
.0062	87.69	71.16	76.94	.0162	134.39	97.15	103.38
.0064	90.37	71.97	77.78	.0164	134.83	97.49	103.72
.0066	93.04	72.76	78.60	.0166	135.27	97.83	104.06
.0068	95.71	73.53	79.40	.0168	135.70	98.16	104.39
.0070	98.37	74.28	80.17	.0170	136.13	98.50	104.72
.0072	101.02	75.01	80.93	.0172	136.55	98.82	105.04
.0074	103.68	75.73	81.67	.0174	136.97	99.15	105.36
.0076	106.32	76.43	82.39	.0176	137.38	99.47	105.68
.00769	*107.53*						
.0078	108.02	77.11	83.09	.0178	137.79	99.78	105.99
.0080	108.92	77.78	83.78	.0180	138.19	100.09	106.30
.0082	109.79	78.44	84.46	.0182	138.59	100.40	106.60
.0084	110.65	79.08	85.11	.0184	138.98	100.71	106.91
.0086	111.49	79.71	85.76	.0186	139.37	101.01	107.20
.0088	112.31	80.33	86.39	.0188	139.75	101.31	107.50
.0090	113.11	80.93	87.01	.0190	140.13	101.60	107.79
.0092	113.90	81.52	87.61	.0192	140.50	101.89	108.08
.0094	114.67	82.10	88.21	.0194	140.87	102.18	108.36
.0096	115.43	82.67	88.79	.0196	141.24	102.47	108.65
.0098	116.17	83.23	89.36	.0198	141.60	102.75	108.92
.0100	116.90	83.78	89.92	.0200	141.96	103.03	109.20

FOUNDATIONS

(PART 1)

INTRODUCTION

1. Definitions.—It is obvious that the weight of any artificial structure must be, directly or indirectly, transferred to and carried by some part of the natural surface of the earth. The word *foundation*, as used in connection with engineering structures, is commonly applied both to the natural material, or the portion of the earth's surface, on which a structure rests, and to the lower part of the structure itself that is in contact with or contiguous to the natural surface. To prevent the confusion likely to result from this indefinite use of the word, various distinguishing terms have been employed. Some writers call the natural surface on which the structure rests the **foundation bed.** Here, however, the term *subfoundation*, which seems appropriate, will be used instead of *foundation bed*, and the following definitions will be adopted:

The **subfoundation** of a structure is that part of the natural surface of the earth on which the structure rests.

The **foundation** of a structure is not only that part of the structure that is directly in contact with the subfoundation, but also as much of the lower part of the structure as may have to be modified to properly connect the structure with the subfoundation. Thus, if a bridge pier rests on level solid rock, equal in strength to the material of which the pier itself is composed, it may be safely designed as shown in Fig. 1, in which case the foundation is restricted to, at most, the

first course of masonry. But if it is necessary to distribute the weight of the structure over a larger area of subfoundation, as much of the lower part of the structure as must be modified to comply with this requirement may be properly called a part of the foundation. Thus, in Fig. 2, all that part of the structure from *a* up to *b* may be properly called the foundation.

The definition, sometimes given, that the foundation is

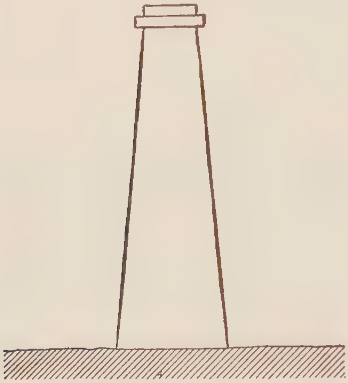


FIG. 1

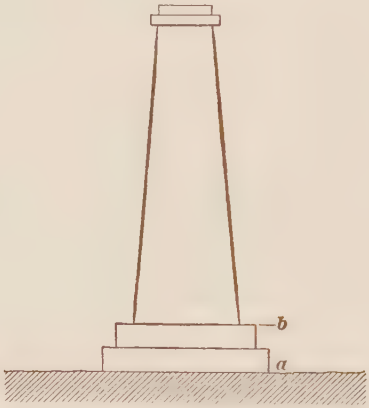


FIG. 2

that part of a structure that lies below the natural surface of the ground, or below the surface of the water, is indefinite and inaccurate.

2. Main Factors Governing the Design of Foundations.—The stability and life of an engineering structure depend largely on the character of its foundation. The selection and designing of suitable foundations is, therefore, one of the most important and responsible duties of engineers. The requirement of first importance is that the foundation should be of sufficient strength to support permanently the proposed structure. On the other hand, sound economy requires that no more money should be spent on the foundation than intelligent skill and prudence may dictate. It is not sufficient that the engineer may be able to show that a foundation designed by him is unquestionably secure; he

should also be able to prove that an adequate foundation could not have been provided with less expenditure of money and time.

The great variety of materials and conditions that are met with in foundation work calls for the exercise of good sense and sound judgment, as well as for the application of technical knowledge. Foundation problems do not always lend themselves to exact mathematical solution to the same degree as, for instance, the designing of a bridge truss, where the properties of the steel are comparatively uniform and well known, and the magnitude and distribution of all the stresses can be determined with precision. It is none the less true, however, that the intelligent solution of a problem relating to foundations requires a thorough knowledge of mechanical principles, and the ability to apply them properly.

THE SUBFOUNDATION

MATERIALS

CLASSIFICATION

3. The subfoundation of a structure may consist of any of the materials that are found on the earth's surface and are suitable for the purpose. The materials usually regarded as suitable are: (1) *solid rock*, including shale in its natural geological position; (2) *loose rock*, which is rock broken into masses of comparatively small sizes; (3) *earth*, including clays, loams, and bogs, in whatever condition they may be found; (4) *sand*, including gravel.

In some cases, two or more, or even all, of these materials may be met with in the same subfoundation.

ROCK SUBFOUNDATION

4. Rock in its undisturbed geological position forms the best subfoundation, and is always to be preferred when it is available. For important structures of great weight, rock

subfoundations are considered almost indispensable. Nearly any sound durable rock, including shale of average hardness, found in its natural position, may be safely used for the support of any ordinary structure.

5. Crushing Strength of Solid Rock.—The supporting power of a rock subfoundation may be considered as approximately equal to the resistance to crushing of the material of which the rock is composed, modified by a suitable factor of safety. Samples selected for crushing tests are likely to be superior to the average, and as, in large areas, imperfections are likely to exist, the factor of safety should be a liberal one: it should not ordinarily be less

TABLE I

CRUSHING STRENGTH OF ROCK

Kind of Rock	Ultimate Crushing Strength					Safe Foundation Load, Tons per Square Foot Factor of Safety of 10		
	Pounds per Square Inch		Tons per Square Foot					
	From	To	From	To	Average	From	To	Average
Granite . .	10,000	20,000	720	1,440	1,080	72	144	108
Limestone .	6,000	18,000	430	1,300	870	43	130	87
Sandstone .	4,000	15,000	300	1,080	690	30	108	69
Shale . . .	400	14,000	30	1,010	520	3	100	52

than 8, nor, except in rare cases, more than 15. If the rock appears to be of uniform good quality, without cracks or seams, over and for a considerable space around the area to be occupied by the foundation, a load equal to one-tenth of the ultimate crushing strength of the material may be confidently used, subject to the precautions hereafter mentioned. Crushing tests are usually made on small cubes, and the crushing strength is stated in pounds per square inch of the surface exposed to pressure. The units commonly applied to foundations are the square foot and the ton (2,000 pounds), and it is usually assumed that the strength per square foot of any material is 144 times the crushing strength per square inch.

The crushing strength of the different kinds of rock, as well as of different varieties of the same rock, varies within wide limits. Table I gives the approximate crushing strength of the various kinds of rock more frequently met with, and the foundation loads that they may usually be depended on to carry safely under normal conditions.

LOOSE-ROCK SUBFOUNDATIONS

6. Description of Loose Rock.—Under the operation of natural laws, the solid strata or deposits of rock have, in many cases, been broken up, and the resulting débris moved from their original position by the action of gravity, water, and ice. The individual masses of this broken-up material may be of any size or form. Masses of considerable size and angular shape are commonly called **loose rock**, while smaller fragments that have become rounded by attrition are called **boulders**. In a deposit of such materials, there will exist cavities between the separate masses of rock, which may be voids or may be filled with smaller fragments of the same material, or with earth or sand. If filled with a material that has hardened into rock and has cemented the fragments into a solid mass, the rock is called **breccia, conglomerate,** or **pudding stone.**

7. Loose Rock as a Subfoundation.—Loose rock in any of its forms may make a satisfactory subfoundation. The celebrated cantilever bridge at Niagara Falls rests on a loose-rock subfoundation composed of large masses of rock detached from the adjoining cliffs, the spaces between the large fragments of rock being filled with smaller fragments and with earth. The pressure on this subfoundation is about 3 tons per square foot. Subfoundations of this kind, owing to the comparatively recent geological age of the formations and the conditions under which the deposits have been made, require careful examination, and, if possible, should be avoided for important structures.

EARTH SUBFOUNDATIONS

8. General Description.—Under the general head of earth are here included the different clays, loams, and other soils, in whatever condition they may be found. In hardness and capacity to sustain weight, they vary all the way from the indurated clays and hard pans to soft mud, often mixed with more or less of the organic matter found in swamps and bogs. Nearly all these materials are porous in structure, and, when confined and subjected to sufficient pressure, are compressed to a greater or less degree. When moist or wet, they are generally *plastic;* that is, they possess the property of yielding or flowing when exposed to pressures exceeding certain limits, these limits being different for different materials. This property of yielding under pressure is so characteristic of the class of materials under consideration, that the term "compressible materials" is generally applied to them in connection with foundation work. The different degrees to which these materials are compressible, and the difficulties and dangers that this property introduces into the problems of earth subfoundations, make them a subject of great interest and importance to the engineer.

It sometimes becomes necessary, when solid rock is inaccessible, to found important structures on clay or earth of a quality far from satisfactory to the engineer, and he will often find his knowledge, experience, ingenuity, and skill taxed to the utmost by the problems presented. For the less important, and in fact for the great majority of ordinary structures, earth subfoundations, if intelligently made use of, may be safely relied on.

9. Strength.—When exposed to increasing pressure, as in testing machines, clay and earth usually yield gradually by flow and deformation of the mass, rather than by distinct crushing, and for this reason crushing tests on small samples are of comparatively little value. Their strength is largely affected by the quantity of water they contain; and the extent to which they may be exposed to water in the subfoundations

is an important element to be considered in determining their sustaining capacity.

The data as to the weight that may safely be placed on earth subfoundations are derived largely from observation and experience. Indurated clay, called **hard pan,** and ordinary clays in comparatively dry situations may usually be depended on to carry a weight of from 1.5 to 2 tons per square foot of surface. In some standing structures founded on clay, the weight on the subfoundation is greatly in excess of these figures, and no settlement or indications of failure have been observed. However, excessive loads on clay subfoundations, even where tests indicate that the structures may be safe, are not sanctioned by good practice.

TABLE II

SAFE LOADS ON EARTH SUBFOUNDATIONS

Kinds of Material	Loads in Tons per Square Foot	
	From	To
Hard pan and other indurated clays	2	$2\frac{1}{2}$
Ordinary clays and clay soils, not submerged in water	$1\frac{1}{2}$	2
Clay, soft and plastic	$\frac{1}{2}$	1
Ordinary soils, comparatively dry	1	$1\frac{1}{2}$
Ordinary soils, wet	$\frac{1}{4}$	1
Swamp and bog material	$\frac{1}{8}$	$\frac{1}{2}$

Experience seems to indicate that the strength of earth subfoundations of the various general kinds may be taken approximately as given in Table II. Owing, however, to the greatly varying individual character of materials that may be classed under the same general name, the engineer must in each case be guided largely by judgment based on experience and actual tests.

SAND SUBFOUNDATIONS

10. Ordinary Sand and Gravel.—Sand and gravel are but slightly compressible, even when saturated with water, and are capable of carrying very great loads. The high

frictional resistance of the separate grains or pebbles on each other tends to prevent the movement or flow that is characteristic of clay. The material is, however, readily eroded by flowing water, and, when utilized for subfoundations, great care must be taken to protect it from direct contact with currents of water, whether of ordinary streams or those caused by wave action. Sand and gravel are both likely to undergo slight initial compression when subjected to pressure; this fact should be taken into consideration, and proper allowance should be made for it in designing structures to rest on this kind of subfoundation. Clean dry sand can bear a load of from 2 to 4 tons per square foot.

11. Quicksand.—One of the most treacherous and troublesome materials with which the engineer has to deal in foundation work is **quicksand.** It is difficult to give a satisfactory definition of this material, though its physical properties are easily described. In most respects it does not differ essentially from ordinary fine sand, and when removed from its native bed and freed from water it cannot usually be distinguished from sands that do not possess its peculiar properties. The separate grains are always small and generally more rounded than those of ordinary sand. But when saturated with water, a mass of this material seems to lose more or less of its internal cohesion or friction, and its power to support weights is almost zero. A bar may be worked to the bottom of a bed of quicksand several feet deep with very slight exertion, and men and animals attempting to walk over such a deposit are often hopelessly engulfed. This description applies to well-developed examples of the material. There is no well-defined demarcation between ordinary sand and quicksand, and the one may pass into the other by imperceptible stages; in other words, sands may possess the peculiar properties of quicksand in very different degrees.

In general, quicksand has practically little or no value as a subfoundation; however, it may sometimes be so treated as to make it serve to support ordinary structures. If a bed

of quicksand is drained, it loses its distinctive property; it becomes more compact and resisting than ordinary sand, and may make a convenient and secure subfoundation.

SUBFOUNDATIONS OF MIXED MATERIALS

12. It is often found that the site of a structure is occupied by several of the materials described—either in separate deposits or mixed together in various proportions. If rock is found, it may extend over only a part of the area of the foundation, or it may contain large fissures filled with other materials; or the rock, although extending over the whole area, may not be all of the same kind or quality, in which case different parts of the subfoundation require different methods of treatment. Again, the area may be occupied by a mixture of earth and loose rock or boulders, or part of it may be clay and part of it sand or quicksand. Such complications are of frequent occurrence, and conditions are always likely to be found that are without precedent in the engineer's experience, and that will tax to the utmost his judgment and skill.

EXAMINATION AND TESTS OF SUBFOUNDATIONS

13. It is generally unsafe to trust to surface appearances in judging of the character, strength, and soundness of subfoundations. A stratum of rock may, when its surface is uncovered, appear to be of satisfactory character, without serious fissures or other defects, and may give the impression of being continuous and solid downwards to an indefinite depth. This appearance may, however, be deceptive; the layer of rock may be comparatively thin, and may be underlaid with a body of soft clay or other unresisting material, and may therefore not be capable of sustaining the structure designed to rest on it. Not a few important engineering structures have failed from the existence of such unforeseen conditions. Likewise, an apparently satisfactory bed of clay may be too thin, or may be underlaid with a stratum of sand that the swift current of a stream may sometime reach and

carry away, undermining the clay and causing the structure founded on it to fail. It is therefore necessary that the engineer should carefully investigate not only the material that he purposes to use for a subfoundation, but, as far as may be possible, all the conditions surrounding it, and satisfy himself that these conditions are not such as may ultimately result in the destruction or impairment of the proposed structure.

EXAMINATION AND TESTS OF ROCK SUBFOUNDATIONS

14. Geological Examination.—A general knowledge of the geology of the region—such as the character and succession of the local formations, the inclination or dip of the strata, and the prevalence of faults and other disturbances—should be acquired, and this knowledge should be utilized in the study of the immediate locality of the work. In a region where the geological formations have been subjected to upheaval and are greatly broken up or much inclined, greater caution is necessary than where the strata remain in their original nearly horizontal and unbroken condition.

Where the structures to be erected are of secondary importance and the weight to be carried is comparatively small, the character of the subfoundation may often be determined accurately enough from the nature of the neighboring strata and their outcrop in near-by streams or ravines. For all important works, however, special tests should be made.

15. Drill Test and Drilling Machines.—Where the subfoundation is rock, it may be most satisfactorily examined by sinking drill holes into it. The number and depth of these drill holes depend on the apparent conditions and the importance of the structure. In the case, for instance, of a bridge with short spans carried on comparatively low piers, the same thoroughness of investigation is not called for as in the case of a long suspension bridge with high piers, carrying very heavy loads. In the former, one or two drill holes on the site of the pier, carried to a depth of 10 feet, or even

less, might be sufficient; while in the latter, drill holes not only within, but for some distance outside of, the boundaries of the pier and carried to a depth of from 20 to 30 feet, or more, may be advisable.

16. In the great majority of cases, drill holes of the size of, and made with the apparatus commonly used for, those in quarry and rock-excavation work, will answer every purpose. In the more important works, particularly where there is reason to expect defects and irregularities, and where the holes are to be carried to a considerable depth, some one of the forms of drilling apparatus that yield complete samples, or **cores**, of the material passed through should be used. These machines are generally power driven. The drilling tool is a hollow rod that, when revolved, makes an annular cut into the rock, leaving an undisturbed section, or core, within the tool; this core is occasionally broken off and brought up with the tool. The recovered cores may, after examination and record, be preserved for future reference.

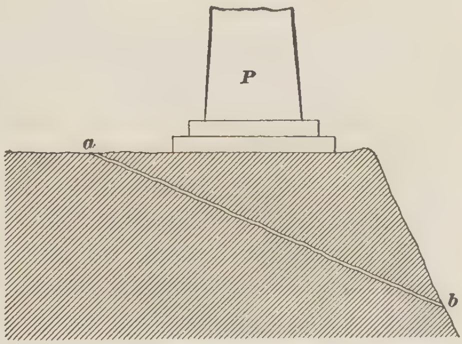


FIG. 3

17. Test drilling will, if properly conducted, disclose any subterranean defects in the subfoundation, and will furnish data for a second judgment of the capacity of the material. The defects that may thus be disclosed are: (1) beds or pockets of clay or other soft material dangerously near the surface; (2) large cavities that might weaken the subfoundation; and (3) thin layers of clay or argillaceous shale, which, in cases where the strata are much inclined and other conditions permit, might allow the structure and its foundation to slide or move laterally. Such a possible condition is illustrated in Fig. 3. The weight of the pier P and its

load might cause the subfoundation to slide along the inclined joint *a b*, particularly if this joint is lubricated by a thin layer of unctuous clay. This is not a purely suppositious case, since the destruction of not a few structures has been caused by similar conditions.

EXAMINATION AND TESTS OF OTHER SUBFOUNDATIONS

18. All that has been said of rock subfoundations applies with equal force to loose rock, earth, and sand subfoundations; in fact, these usually require the exercise of greater care and caution than ordinary rock foundations. The formations consisting of these materials, as they are now found, are much more recent geologically than rock formations, and are far less stable in character. It is not at all uncommon to find beds of firm strong clay that prove comparatively thin and are underlaid with deposits of much softer clay, alluvium, or quicksand, saturated with water and having very little capacity to carry loads. Two methods are in common use for testing such subfoundations; namely, *test pits* and *borings*.

19. Test pits, or wells, are ordinary excavations similar to the common well, carried to such depth as may be thought necessary, within or near the boundary of the foundation. The method of sinking such test pits requires no special description.

20. Borings.—The method of boring is, in general, similar to the rock drilling described in Art. **16,** but is usually less expensive and more expeditious. The bore holes are really wells or test pits of a small diameter. Several methods of making such borings are in use, some of which are so simple as to require no description here. Frequently, the procedure and the apparatus used are almost the same as for rock drilling. This method is objectionable because of the difficulty of obtaining good samples and determining the character of the materials passed through, but it may give results that are satisfactory in many cases

Thus, a drill hole sunk into a bed of clay or other material may show that no important change in its character occurs within such a distance from the surface as would render it unsuitable for foundation purposes. But even if the results from this method of making borings were always satisfactory, difficulties are frequently met with that make it inapplicable. A layer of sand or other soft material may be encountered that has not sufficient stability to maintain the wall of the bore hole; the material caves or runs into the hole, filling it and obstructing the operation of the drill. To prevent this, the hole must be *cased;* this is done by driving down, inside the hole, an iron tube. When borings are to be carried to a considerable depth, it is generally best to begin with the expectation that casing will be necessary, and to make the bore hole large enough to allow the use of iron pipe not less than 3 inches inside diameter.

21. For these larger holes, two well-defined methods of boring are employed. In the one, suitable earth augers are used, which excavate and bring up the material in its natural condition; in the other, called **wash boring,** the material is loosened partly by the auger or drill, and partly by a jet or current of water, which also carries the loosened material to the surface. In this method, the drill rod is tubular, and a current of water is forced through it by pumps or other devices. The water escapes at the working end of the drill and rises through the annular space between the drill and the casing, carrying with it the excavated material, which may be collected and examined. As the drill progresses downwards, the casing is, from time to time, forced downwards by a pile driver or other suitable device, additional lengths of pipe being added by means of screw joints or couplings, as required. The first method yields the most satisfactory samples of the material passed through; the second is more expeditious, and, with a proper equipment, more economical. Whatever method may be employed, the results of borings should be carefully observed, accurately recorded, and samples preserved for future reference.

DESIGN OF THE SUBFOUNDATION: THEORY OF PRESSURE

22. Required Area of Subfoundation.—In the case of foundations for ordinary structures, where weight is the only force to be resisted, and where that weight is uniformly distributed over the whole of the subfoundation, the first problem of importance will be: Over what area of the subfoundation must the weight be distributed, in order that the safe bearing capacity of the material of the subfoundation shall not be exceeded?

Where the material is solid rock in its natural geological position, it is usually unnecessary to solve this problem, since almost any sound rock is sufficiently strong to carry the concentrated weight of any well-designed structure. But where earth or sand subfoundations must be utilized, care should be taken to distribute the weight to be carried over such an area that the resistance of the material shall safely exceed the applied weight. Thus, if the estimated weight of a bridge pier is 1,200 tons, the weight that may come on it from the dead load of the bridge, 500 tons, and the weight of expected live load, 300 tons, the subfoundation will be depended on to sustain a load of 2,000 tons. If the subfoundation is clay and may be safely loaded with 2 tons per square foot, the area required will evidently be $2{,}000 \div 2 = 1{,}000$ square feet.

EXAMPLE.—The foundation for a stand pipe has the form of a frustum of a cone, the dimensions being as follows: diameter of upper base, 24 feet; diameter of lower base, 30 feet; height, 5 feet. The weight of the material of the foundation is 100 pounds per cubic foot, and the weight of the stand pipe is 500 tons. If the subfoundation is clay that may be loaded with 2 tons per square foot, what is the required area?

SOLUTION.—The weight of the foundation is (see *Geometry*, Part 2)

$$\left(24^2 \times .7854 + 30^2 \times .7854 + \sqrt{24^2 \times .7854 \times 30^2 \times .7854}\right) \times \frac{5}{3} \times 100$$
$$= 287{,}460 \text{ lb.} = 143.73 \text{ T.}$$

The total load on the subfoundation is $500 + 143.73 = $ **643.73 T**

The required area of the subfoundation is, therefore,

$$643.73 \div 2 = 321.87 \text{ sq. ft., nearly. Ans.}$$

23. If the plan of the structure is irregular, so that it is difficult to secure a uniform intensity of pressure over the whole subfoundation area, or if lateral forces must be taken into consideration, or if the character or sustaining capacity of the material of the subfoundation varies over parts of it, the question becomes more complicated. Clay and earth subfoundations are usually more or less compressible; the amount of compression or settling is in direct proportion to the intensity of the applied pressure, and a slight variation in the settlement of different parts of the subfoundation may distort or disrupt the structure erected on it.

A not uncommon example of a load not uniformly distributed is a masonry arch culvert built on a compressible subfoundation under a high railroad embankment, as illustrated

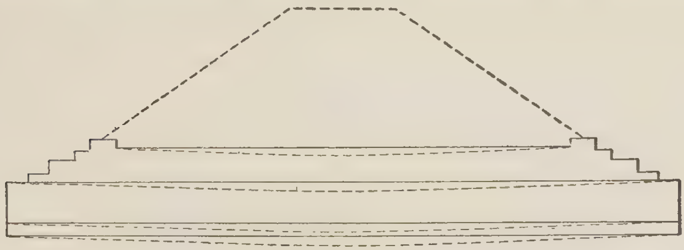


Fig. 4

in Fig. 4. The weight on the subfoundation varies with the height of the embankment over it, being greatest directly under the middle. If the foundation of such a culvert is made of the same width throughout, the intensity of pressure on it will increase as the middle section is approached, unequal settlement may follow, the structure may take the distorted form shown by the dotted lines, and the distortion may be sufficient to fracture the masonry. If the area of the subfoundation is increased properly from the ends toward the middle of the structure, so as to distribute the load evenly per unit area, then, whatever settlement may take place will be the same throughout the whole length of the structure, and the masonry will not be injured. The walls of heavy buildings are often cracked through the failure of the designer to properly proportion the area of the subfoundation to the pressure on it

It is always desirable to avoid subfoundations of compressible material, or, if this cannot be done, to make the loading so light that settlement, if any, will be so slight as to be negligible. But as this is not always possible, and as it can seldom be determined in advance whether a material will be compressed under a given load, and, if so, how much, it is important that in earth subfoundations every possible effort should be made to distribute the pressure as evenly as possible over the whole area covered.

24. Depth of Subfoundation Below Surface of Ground.—Foundations in earth should be carried to such a depth below the ground surface that frost will not reach them. Nearly all moist earth expands, or heaves, with freezing, and repeated freezing is likely to soften and disintegrate it. It may also be subject to other disturbances near the surface. The depth of foundations may be dictated by conditions other than frost. Often a good material cannot be found except at greater depths than are necessary to provide against frost.

The penetration of frost varies with the latitude. In the American Gulf States, ice seldom forms; while in the Lake region, the ground sometimes freezes to a depth of 5 or even 6 feet. Ordinarily, in the northern parts of the United States, subfoundations 4 feet below the ground surface may be considered safe from injury by frost.

25. Effect of Weight and Friction of Superimposed Earth.—Not only is the soil likely to be firmer and harder a few feet below the surface, but the surrounding earth, by its weight, tends to counteract any movement and therefore to increase the bearing capacity of the subfoundation. Thus, if a wall is founded at a depth of 5 feet below the surface, the weight of the surrounding earth above the subfoundation tends to prevent any flow of the clay from under the foundation. Such movement of the material can only take place by overcoming the weight and internal friction of the banks of clay around the foundation, resulting in the upheaval of the surface around the structure.

This counter pressure of the superimposed earth is a factor of much importance in the strength of subfoundations in plastic soil, but its exact value is difficult to compute. If the soil were a fluid, like water, the upward pressure under the foundation would be equal to the weight of a column of fluid having a height equal to the depth of the subfoundation below the surface and a cross-section equal to the area of the foundation. But, in clay, this pressure is modified by the internal friction of the material. The problem of pressures in plastic or semifluid materials like clay has been studied by physicists, and formulas to express its value have been devised, but the results do not seem to agree closely with observed facts, and these formulas must therefore be used with caution.

Rankine's formula is perhaps most often referred to by engineers. It is as follows:

$$p = w\,h\left(\frac{1 + \sin Z}{1 - \sin Z}\right)^2$$

in which p = intensity of pressure that the material can bear;

w = weight of material per unit of volume;

h = depth of subfoundation below surface;

Z = angle of repose of the material.

If h is in feet and w in pounds per cubic foot, p will be in pounds per square foot.

It should be borne in mind that Rankine's formula, although the best that has been proposed for this purpose, is but a rough approximation, and should be used only as a guide. In all important work, the bearing capacity of the soil should be ascertained by direct experiment. Thus, to determine the safe bearing capacity of the clay in which the walls of the Capitol at Albany, New York, are founded, rectangular pits about 3 feet square and 3 feet deep were excavated, and in them were built columns of masonry 3 feet square, on which various trial loads were placed. Under a load of 2 tons to the square foot, the clay subfoundation was slightly compressed, but no indications of flow were observed. When the load was increased to 6 tons per square foot, the

clay under the pillar yielded very perceptibly and began to heave up around the pillar, as shown by the dotted line in Fig. 5.

The facts just explained illustrate a point that seems to be often overlooked in designing heavily loaded foundations in compressible soils. Referring to Fig. 6, which represents a structure founded on clay on a sloping hillside, it is evident that the high subfoundation wall of clay on the upper side of the structure will exert a greater pressure to counteract the flow of the clay from under the foundation than will the low wall on the lower side; that, under loads that might cause yielding on the lower side, the foundation might settle there, while remaining firm on the upper side. In such situations, care should be taken to keep the loads on the whole of the foundation within such safe limits that the clay where least confined or counterbalanced will not yield by flowing. If, for any reason, this is not possible, and loads of questionable magnitude must be used, the danger may be averted by dressing down the high bank to a slope less than the angle of repose of the material, as illustrated by the dotted line in Fig. 6. There is reason to believe that in not a few cases where heavily loaded foundations have settled unevenly, this has been due to the unequal counterbalance afforded by the superimposed subfoundation walls.

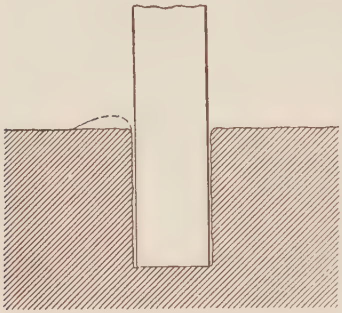


Fig. 5

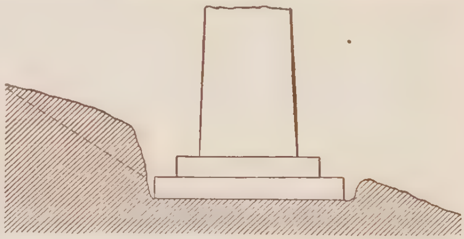


Fig. 6

Example 1.—Taking the angle of repose of a certain semifluid soil as 31°, and the weight as 100 pounds per cubic foot, what must,

according to Rankine's formula, be the depth of the subfoundation below the surface that its bearing value may be 4 tons per square foot?

SOLUTION.—The formula of Art. **25**, solved for h, gives

$$h = \frac{p}{w}\left(\frac{1 - \sin Z}{1 + \sin Z}\right)^2$$

Here, $p = 4$ T. per sq. ft.; $w = 100 \div 2{,}000 = .05$ T. per cu. ft.; $Z = 31°$; $\sin Z = .515$. Therefore,

$$h = \frac{4}{.05}\left(\frac{1 - .515}{1 + .515}\right)^2 = 8.20 \text{ ft. Ans.}$$

EXAMPLE 2.—The depth of a subfoundation is 12 feet below the surface. (*a*) If the angle of repose of the material is $14\frac{1}{2}°$, and the weight is 110 pounds per cubic foot, what is the bearing value of the material? (*b*) If the weight of the structure, including the foundation, is 350 tons, what is the area required?

SOLUTION.—(*a*) To apply the formula of Art. **25**, we have $w = 110$; $h = 12$; $\sin Z = \sin 14° 30' = .250$. Substituting in the formula,

$$p = 110 \times 12\left(\frac{1 + .250}{1 - .250}\right)^2 = 3{,}667 \text{ lb. per sq. ft. Ans.}$$

(*b*) The weight of the structure is 350 T.; therefore, the area required is

$$\frac{350 \times 2{,}000}{3{,}667} = 191 \text{ sq. ft. Ans.}$$

EXAMPLES FOR PRACTICE

1. The angle of repose of a certain soil is 20°, and its weight is 100 pounds per cubic foot. What must be the depth of the subfoundation below the surface that its bearing value may be 3 tons per square foot? Ans. 14.42 ft.

2. The angle of repose of a certain soil being 25°, its weight, 110 pounds per cubic foot, and the depth of the subfoundation 10 feet, what will be its bearing value? Ans. 6,659 lb. per sq. ft.

3. The angle of repose is 30°; the weight of the soil, 100 pounds per cubic foot; and the depth of the subfoundation, 8 feet. (*a*) What is the bearing value of the subfoundation? (*b*) If the weight of the structure, including the foundation, is 300 tons, what is the area required? Ans. $\begin{cases} (a) \text{ 3.6 T. per sq. ft.} \\ (b) \text{ 83 sq. ft.} \end{cases}$

26. Effect of Lateral Forces.—Many important engineering structures—particularly dams, retaining walls, and abutments—are subjected to lateral forces in addition to weight. These may change the magnitude, and always

change the distribution, of the total pressure on the subfoundation. A dam may serve as an illustration. Here, the forces that act on the structure are the weight of the structure itself and the pressure of the water against its inner face.

Let $ABED$, Fig. 7, be a cross-section of a dam resting on the subfoundation AB. For convenience, a portion of the dam 1 foot long will be considered. The center of the base is at c, and the center of gravity of the part considered is at g. The pressure P, normal to the inner wall BE, exerted by the water can be determined by the methods explained in *Hydrostatics;* also, the point of application or line of action of P. The resultant R of the pressure P and the weight W of the structure gives the resultant pressure acting on the subfoundation; its line of action ad meets the subfoundation at e, which is the **center of pressure,** or point of application of the resultant pressure. If the center of pressure were at c, the center of the base, the weight would obviously be evenly distributed over the whole subfoundation. But with the center of pressure moved to e, the pressure will not be uniform; it will obviously be greater at A than at B.

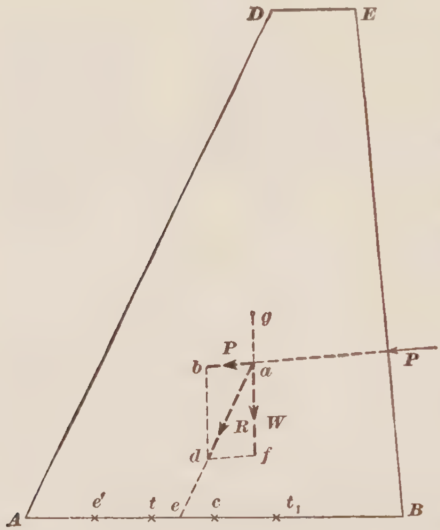


FIG. 7

27. Rule of the Middle Third.—If the base AB, Fig. 7, is divided into three equal parts At, tt_1, t_1B, the center part tt_1 is called the **middle third.** It is a principle of the statics of structures and foundations that, in order that neither end of the subfoundation or of the structure

itself may be subjected to an uplifting force, the center of pressure for a rectangular subfoundation should fall within the middle third of the subfoundation. If the center of pressure is outside the middle third, as at e', Fig. 7, the farther end B of the base will have a tendency to rise; there will, therefore, be tension instead of compression at B—a condition that should not obtain in masonry construction.

28. Intensity of Pressure.—In general, let AB, Fig. 8, be a rectangular subfoundation, or a course of masonry on which a structure or part of a structure rests. The base AB, whose length, in feet, is denoted by L, is trisected at t and t_1, and bisected at C. Let V (pounds) be the vertical component of the total pressure acting on AB; and let the distance eC of the point of application of this pressure from the center C be denoted by d. Then, the greatest intensity of pressure p_1, in pounds per square foot, will occur at A, and the least intensity p_2 will occur at B. It can be shown by the use of advanced mathematics that p_1 and p_2 are given by the formulas:

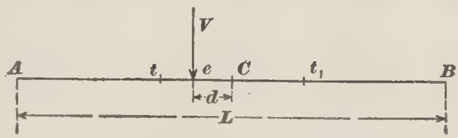


FIG. 8

$$p_1 = \frac{V}{L} + \frac{6\,Vd}{L^2} \qquad (1)$$

$$p_2 = \frac{V}{L} - \frac{6\,Vd}{L^2} \qquad (2)$$

The condition for which there is no pressure at B is obtained by making $p_2 = 0$. Putting the second member of formula 2 equal to zero, and solving for $\frac{V}{L}$, there results

$$\frac{V}{L} = \frac{6\,Vd}{L^2};$$

whence

$$d = \frac{L}{6},$$

and, therefore,

$$Ae = \frac{L}{2} - d = \frac{L}{2} - \frac{L}{6} = \frac{L}{3} = At$$

That is, the point e coincides with t. If d is greater than $\frac{L}{6}$, or $A e$ less than $\frac{L}{3}$, $\frac{6\,V d}{L^2}$ is greater than $\frac{V}{L}$, and p_2 becomes negative, which indicates tension instead of pressure at B. Hence the middle-third rule given in Art. **27.**

PREPARATION OF THE SUBFOUNDATION

29. Rock Subfoundation.—The preparation of the subfoundation is generally a very simple matter, involving only manual labor. In rock subfoundations, it is not usually considered necessary to excavate below the surface to a greater depth than may be found needful to remove disintegrated and weather-worn rock, and to bring the surface into proper shape to receive the foundation. Where a stone-masonry structure is to rest directly on a sloping-rock subfoundation, the latter must be leveled off to receive the courses of masonry, or else cut into benches or terraces, according to circumstances. The dressing of the rock into benches may sometimes be avoided by the use of concrete for leveling up inequalities between the rock and the masonry, as illustrated in Fig. 9. This is, however, permissible only where the general slope of the rock is not sufficient to induce or permit sliding of the concrete on the surface of the bed rock, caused by gravity or by lateral forces acting in the direction of the downward inclination. In the case of dams, retaining walls, abutments, etc., subjected to great lateral stress and liable to failure by sliding on the subfoundation, sloping surfaces must be avoided. One of the rules commonly laid down by writers on foundations is that the surface of the subfoundation should be perpendicular to the resultant of all the forces acting on the structure. This rule

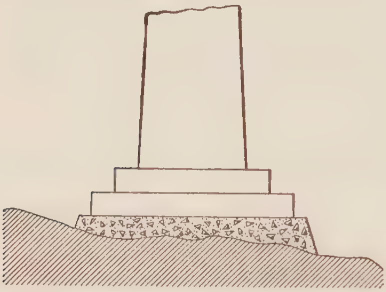


FIG. 9

is correct in principle, but it may often be deviated from within safe limits for economic reasons, as will be seen further on.

30. Earth Subfoundation.—In the preparation of earth subfoundations, the material is excavated to a sufficient depth to guard against the action of severe frost, and the whole area is graded to a level plane. In very soft and compressible material, measures are sometimes taken to solidify or reinforce the subfoundation before the structure is begun. This may be accomplished by driving a large number of short wooden piles over the area, to compress the soil into a more compact mass; but, unless these piles are below the permanent surface of the water, they may in time decay and become useless. To avoid this possibility. piles are sometimes driven and at once pulled out, and the holes are filled up with clean sand or gravel. Sand, dry earth, or crushed rock spread over the surface is sometimes used, but even if rammed into the soft material, they are not likely to prove satisfactory under heavy structures, as the comparatively thin and flexible layer between the soft material and the base of the structure may not sufficiently prevent the shifting of the soft material below.

31. The effect of a deep excavation on the bearing capacity of soft and plastic soils has been referred to in Art. **25**, and very satisfactory subfoundations for ordinary structures may often be thus secured in material that, at or near the surface, would have little supporting power. Deep excavations for foundations are sometimes partly refilled with sand or gravel. This gives good results, which are probably due to the counterpressure caused by the sand or gravel on the soft material. Artificial support is sometimes provided by enclosing the area to be occupied by piles or other retaining structures. Some of these methods will be referred to later.

THEORY OF FOUNDATIONS

FOOTINGS

32. Necessity for Footings.—The office of the foundation is to form a suitable connection between a structure and the subfoundation. If, as is sometimes the case, the material of the subfoundation is equal in strength, hardness, and durability to the material of which the structure itself is built, no foundation is required in the sense in which the word is here used, except in so far as the name may be applied to the line or surface of contact between the natural material of the subfoundation and the more or less artificially prepared material of the structure. It is common to construct the lower courses of piers, walls, and like structures, with steps or projections, as shown in Fig. 10, but these projections have no utilitarian value if the material of the subfoundation is equal in strength and durability to that of the structure. Thus, in Fig. 10, if the structure is of granite or limestone and the subfoundation is of the same or equally good material, the offsets in the footing courses might safely be dispensed with, and the lines of the structure continued down to the subfoundation, as shown by the dotted lines. These projecting courses may, however, possess a certain esthetic value, which justifies their use.

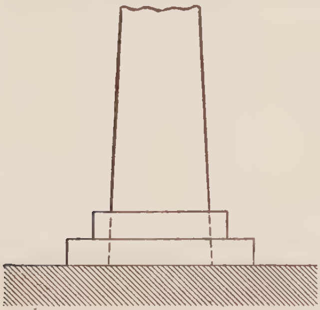


Fig. 10

Where the material of the subfoundation is inferior in strength to the material of the structure, it obviously becomes necessary to accommodate the one to the other, in order to secure equal strength throughout. The material

of the structure may readily carry a load of 20 or more tons per square foot, while that of the subfoundation may be capable of carrying only 2 tons per square foot. The problem is, therefore, to distribute the load carried by the structure over such an area of the subfoundation that its safe load shall not be exceeded. This is ordinarily accomplished by the use of what is commonly called the method of **spread foundations.** Essentially, this method consists in enlarging the lower end or base of the structure so that it may cover an area of subfoundation the aggregate resistance of which will be at least equal to the weight of the structure. The usual method of accomplishing this is by stepping out or enlarging the base of the structure by offsets, called **footing courses,** or simply **footings.** The material employed in these footing courses may be the same as, or different from, that used in the body of the structure, the kind of material used being determined mainly by the conditions that are to be met in each particular case.

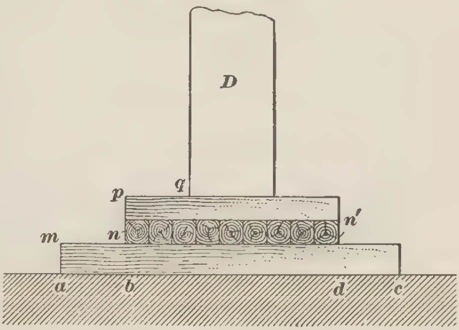


FIG. 11

33. Strength of Footings.—Footings are treated as cantilevers uniformly loaded. To secure the requisite strength most economically, two or more sets of footings are often superimposed to form steps, as shown in Fig. 11; this has the effect of decreasing their loaded free length and enabling them to sustain greater loads. In this figure, which illustrates a timber foundation, there are two footings, $m\,n$ and $p\,q$. The force acting on $m\,n$ is the upward pressure of the part $a\,b$ of the subfoundation; this pressure is assumed to be uniformly distributed, its intensity being equal to the total load on the subfoundation divided by the area of the

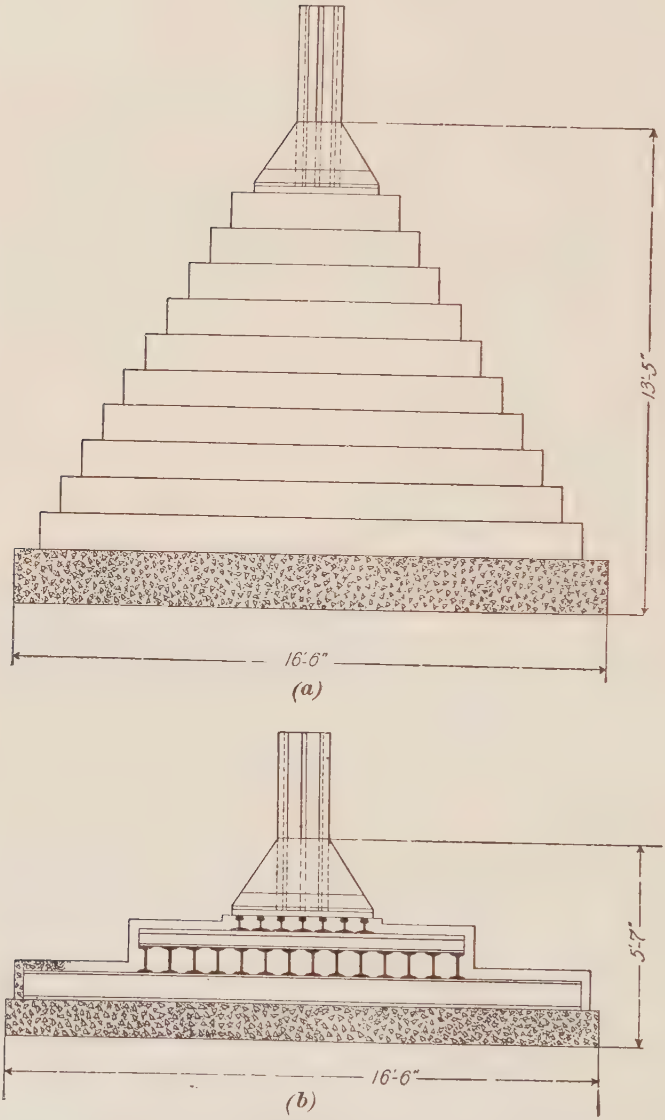


FIG. 12

subfoundation. Likewise, the force acting on pq is the upward pressure, or reaction, of the timbers nn', whose intensity is the total load on the timbers divided by the combined area of their upper or supporting surfaces.

It may be necessary to use several sets of footings; and the result will be to increase greatly the height and volume of the foundation. To obviate this increase, a stronger material, as steel, may be substituted for the beams of wood, in which case one footing may be sufficient. In masonry work, the short projections allowable with cantilevers of stone may necessitate the use of very deep and bulky foundations, and it may be desirable or necessary to substitute steel to reduce this height and bulk. Thus, in Fig. 12, (*a*) illustrates a masonry foundation for a steel column in a large building; if steel beams embedded in concrete are substituted, the foundation may be made much smaller, as shown at (*b*).

34. The formula for the load that can be safely supported by a cantilever beam is

$$W = \frac{2 s_b I}{j l c} \qquad (1)$$

in which W = total distributed load, in pounds;
c = distance from the outermost fiber to the neutral axis;
l = length of beam, in inches;
s_b = modulus of rupture, that is, ultimate bending strength, of material, in pounds per square inch;
I = moment of inertia of the section, referred to the inch;
j = factor of safety.

The value of I for a rectangular beam is $\frac{1}{12} b d^3$, and the value of c is $\frac{1}{2} d$ (see *Strength of Materials*, Part 2). Assuming a value of unity for b, and substituting in formula **1**, these values of I and c, there results

$$W = \frac{s_b d^2}{3 j l} \qquad (2)$$

If the weight, in tons per square foot, is denoted by w, then $w = \frac{W \times 144}{2{,}000\, l}$; whence

$$W = \frac{2{,}000}{144} w\, l = \frac{125}{9} w\, l,$$

and formula **2** becomes

$$\frac{125}{9} w\, l = \frac{s_b\, d^2}{3\, j\, l};$$

whence $$w = \frac{3\, s_b\, d^2}{125\, j\, l^2} = .024 \frac{s_b}{j}\left(\frac{d}{l}\right)^2 \qquad (3)$$

From this equation follows

$$d = l \sqrt{\frac{w\, j}{.024\, s_b}} \qquad (4)$$

35. Moduli of Rupture and Factors of Safety.—For the materials most used in foundation work, the ultimate

TABLE III

MODULI OF RUPTURE AND FACTORS OF SAFETY

Material	Ultimate Modulus of Rupture S_b Pounds per Square Inch	Factor of Safety to be Used
Structural steel	60,000	4 to 8
Cast iron	20,000	8
The stronger woods, oak, Southern pine, etc.	10,000	8 to 10
Ordinary woods, white pine, hemlock, etc.	7,500	8 to 10
Granite of average good quality	1,800	10 to 15
Limestone of average good quality	1,500	10 to 15
Sandstone of average good quality	1,200	10 to 15
Brick of average good quality	500	8 to 10
Concrete 28 days old (1 : 3 : 5)	250	10

moduli of rupture and the factors of safety are commonly taken about as given in Table III. Some of these values are somewhat different from those adopted in other branches of engineering; but it should be constantly borne in mind that all such values are rough averages, more or less uncertain, and that it is this uncertainty that makes it necessary

to use a factor of safety. Scarcely any two investigators arrive at the same conclusion as to the exact strength of a given material; and the student should not be surprised at the differences in the values given in different books or adopted in different branches of the engineering profession.

36. Steel Footings.—In the case of rolled-steel beams (not rectangular in section), the moment of inertia of the section may be taken from such tables as are published by the manufacturers of structural shapes. For this kind of material, 15,000 pounds per square inch may be safely used as a working bending strength for foundations exposed to quiescent loads only.

In the long offsets possible with steel shapes, the deflection of the beam under this loading may be important in some structures, and should be determined by the formulas given in *Strength of Materials*, Part 2.

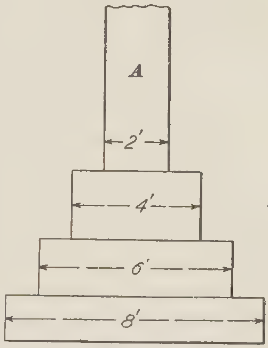


FIG. 13

EXAMPLE 1.—A wall A, Fig. 13, 2 feet thick, carries a load of 12 tons per lineal foot, including its own weight. The foundation of concrete is designed to have each footing project 1 foot beyond the one above. If the subfoundation may be safely loaded with $1\frac{1}{2}$ tons per square foot: (*a*) what should be the width of the base? (*b*) what should be the thickness of each course?

SOLUTION.—(*a*) Since the load is 12 T. per lineal foot and the subfoundation can carry $1\frac{1}{2}$ T. per sq. ft., the width of the base should be

$$12 \div 1.5 = 8 \text{ ft. Ans.}$$

There will, therefore, be three footing courses. (See Fig. 13.)

(*b*) The first course below the wall has a width of 4 ft. and carries a load of $12 \div 4 = 3$ T. per sq. ft.; the second course carries a load of $12 \div 6 = 2$ T. per sq. ft.; and the third course, a load of $12 \div 8 = 1.5$ T. per sq. ft. To apply formula **4**, Art. **34**, to the first footing, we have $l = 12$, $w = 3$, $j = 10$, and $s_b = 250$ (see Table III). Substituting in the formula,

$$d = 12\sqrt{\frac{3 \times 10}{.024 \times 250}} = 26.83 \text{ in. Ans.}$$

For the second footing, $w = 2$, the other values being the same as above; therefore,

$$d = 12\sqrt{\frac{2 \times 10}{.024 \times 250}} = 21.91 \text{ in. Ans.}$$

For the third footing, $w = 1.5$, and

$$d = 12\sqrt{\frac{1.5 \times 10}{.024 \times 250}} = 19 \text{ in., nearly. Ans.}$$

EXAMPLE 2.—A brick foundation is to support a pillar 2 feet square resting on a pedestal $3\frac{1}{2}$ feet square and carrying a load of 150 tons. The capacity of the subfoundation is 1.5 tons per square foot. (*a*) What should be the area of the subfoundation? (*b*) If the foundation is made in four footing courses, each $1\frac{1}{2}$ feet in depth, what should be the dimensions of each course?

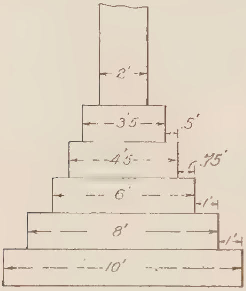


FIG. 14

SOLUTION.—(*a*) The area of the subfoundation should be

$$150 \div 1.5 = 100 \text{ sq. ft. Ans.}$$

(*b*) The base will be made 10 ft. × 10 ft. To determine the allowable length of cantilever for the first course above the subfoundation—that is, the distance the lowest course projects beyond the one above—apply formula **3**, Art. **34**. Solving for l, it is

$$l = d\sqrt{\frac{3\,s_b}{125\,j\,w}}$$

The values of s_b and j taken from Table III are 500 and 10, respectively; $w = 1.5$, and $d = 18$. Substituting in the formula,

$$l = 18\sqrt{\frac{3 \times 500}{125 \times 10 \times 1.5}} = 16 \text{ in., nearly.}$$

The lower course is made to project 1 ft.; therefore, the next course will be 8 ft. × 8 ft. The distributed load is

$$\frac{150}{8 \times 8} = 2.344 \text{ T. per sq. ft.}$$

Substituting the proper values in the formula,

$$l = 18\sqrt{\frac{3 \times 500}{125 \times 10 \times 2.344}} = 13 \text{ in., nearly}$$

This course is made to project 1 ft.; therefore, the next course will be 6 ft. × 6 ft. The distributed load is

$$\frac{150}{6 \times 6} = 4.167 \text{ T. per sq. ft.}$$

Substituting the proper values in the formula,

$$l = 18\sqrt{\frac{3 \times 500}{125 \times 10 \times 4.167}} = 10 \text{ in., nearly}$$

This course is made to project $\frac{3}{4}$ ft.; the next course is, therefore, $4\frac{1}{2}$ ft. $\times$ $4\frac{1}{2}$ ft. The stone pedestal is $3\frac{1}{2}$ ft. square; therefore, the fourth footing course projects $\frac{1}{2}$ ft.

EXAMPLES FOR PRACTICE

1. If the footing courses of the wall in Fig. 13 are composed of granite, what will be the required thickness of each, using a factor of safety of 10?

Ans. Top course, 10 in.; Middle course, 8.16 in.; Bottom course, 7.06 in.

2. A brick foundation is designed to carry a load of 150 tons. (*a*) If the capacity of the subfoundation is 1.5 tons per square foot, what should be the area of the bottom course? (*b*) If the foundation is made in three footing courses, each projecting 1 foot beyond the next, what should be the depth of each of the courses?

Ans. (*a*) 100 sq. ft.; (*b*) Top course, 22.4 in.; Middle course, 16.8 in.; Bottom course, 13.4 in.

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37. Special Cases.—The assumption that the footing courses in foundations may be treated as uniformly loaded

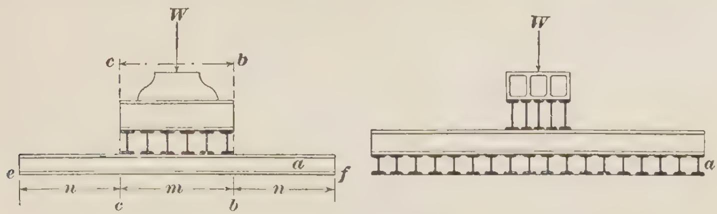
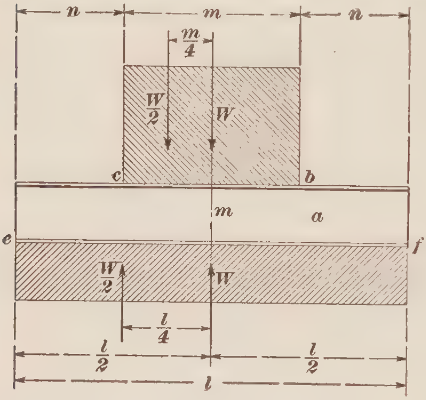


Fig. 15*a*

cantilever beams firmly fixed at one end is not always valid, and the exceptions are so common that they must be considered. In the foundation shown in Fig. 15*a* the total load W from the column may be considered as distributed over the portion $c\,b$ of the top of the beams a, while the upward reactions acting on the bottom of these beams, and also of the amount equal to W, are distributed over the entire sur-

face $c\,f$, the bottom area $c\,b$ receiving only a portion of the load W. This loading is diagrammatically shown in Fig. 15*b*, on which each of the shaded rectangles represents the total load W, the intensity being proportional to the altitudes of these rectangles. The difference in intensity of the loading on the top and bottom of $c\,b$ gives rise to bending stresses in that portion of the beam, and it can be shown by means of advanced mathematics that the maximum bending moment occurs at a point m, midway between c and b. To find the amount of this bending moment take moments about m, Fig. 15*b*, of all the forces to the left of this section. The sum of the moments of the upward forces is equal to $\frac{W}{2} \times \frac{l}{4}$, and that of the downward forces is negative and equal to $\frac{W}{2} \times \frac{m}{4}$. The algebraic sum is



(b)
FIG. 15*b*

$$M = \frac{W}{2} \times \frac{l}{4} - \frac{W}{2} \times \frac{m}{4} = \frac{W}{4}\left(\frac{l}{2} - \frac{m}{2}\right)$$

or, noting that $\frac{l}{2} - \frac{m}{2} = n$, we have

$$M = \frac{Wn}{4}$$

EXAMPLE.—The total load carried by the bottom course of steel I beams, Fig. 15*a*, is 360,000 pounds; the length of the beams is 10 feet; and the width of the course next above it is 3 feet. (*a*) What is the maximum bending moment? (*b*) What size I beam may be used, assuming an extreme fiber stress of 15,000 pounds per square inch?

SOLUTION.—(*a*) The projection at each end of the bottom course is

$$\frac{10 - 3}{2} = 3\tfrac{1}{2} \text{ ft., or } 42 \text{ in.}$$

There are eighteen I beams in the course; therefore, the load on each is

$$360{,}000 \div 18 = 20{,}000$$

Substituting these values in the formula, gives

$$M = \frac{20{,}000 \times 42}{4} = 210{,}000 \text{ in.-lb.} \quad \text{Ans.}$$

(*b*) Referring to a table of moments of inertia, it is found that the moment of inertia of an 8-in. I beam weighing 18 lb. per foot of length is 56.9. The resisting moment of the beam is therefore

$$\frac{15{,}000 \times 56.9}{4} = 213{,}375 \text{ in.-lb.};$$

therefore, an 8-in. I beam may be used. Ans.

EXAMPLES FOR PRACTICE

1. If the width of the upper footing, Fig. 15*a*, is 2 feet, the loading, length, and number of beams in the course *a* being the same as in the preceding example, what will be the greatest bending moment?
Ans. 240,000 in.-lb.

2. An 8-inch I beam, weighing 18 pounds per foot, has a moment of inertia of 56.9. Assuming an extreme fiber stress of 18,000 pounds per square inch, would this beam be sufficiently strong for the load in the preceding example? Ans. Yes.

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38. Footing Courses of Reinforced Concrete.—Footing courses of concrete reinforced with steel rods are

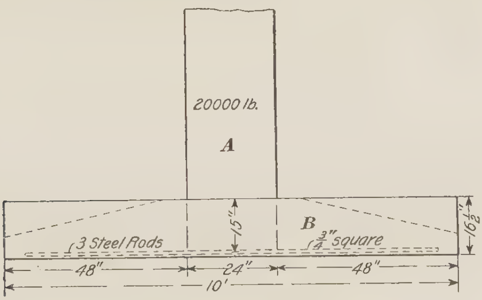


FIG. 16

both efficient and ecomomical. Fig. 16 illustrates a footing of this kind; it consists of a block of concrete into which are built steel rods to resist the tensile stress, the steel being placed as near as possible to the lower surface of the block.

In *Reinforced Concrete*, formulas and tables are included for computing the strength of reinforced-concrete beams for any combination of unit stresses of steel and concrete and for the constants and percentages of steel usually employed in practice. However, for the sake of simplicity, the values recommended by the Joint Committee for certain classes of materials and given in Arts. **69** and **70**, *Reinforced Concrete*, will here be used. Thus, the value of n, which is the ratio of the moduli of elasticity of steel and concrete, will be assumed as 15 and the allowable maximum fiber stresses of steel and concrete, as 16,000 and 650 pounds per square inch, respectively. For these conditions, the formula for the resisting moment in a reinforced-concrete beam is

$$M = R\,b\,d^2 \qquad (1)$$

in which b is the width of the beam, in inches, and d its effective depth—that is, the distance of the center of the steel from the top of the beam—in inches. R is a coefficient depending on the amount of steel used, and its value for different percentages of steel may be taken from columns 2 and 6, Table II, *Reinforced Concrete*. M is the resisting moment of the beam in inch-pounds, and in designing the footing, the resisting moment must be made equal to the maximum bending moment—that is, to the maximum moment of the external forces. Therefore, substituting for M the value derived in Art. **37**, formula **1** becomes.

$$\frac{W\,n}{4} = R\,b\,d^2 \qquad (2)$$

Formula **2** may serve for determining any one of the five quantities involved in the equation when the other four are known. In order to use R, the value of the steel ratio p—that is, the ratio of the area of steel to that of concrete—must be known. As explained in Art. **46**, *Reinforced Concrete*, this ratio is expressed by the formula

$$p = \frac{A}{b\,d} \qquad (3)$$

in which A is the area of steel used in the section of the beam.

In the case shown in Fig. 16, there are assumed to be no stresses on the footing course in a direction parallel to the

wall; but where structures, such as the pillar shown in Fig. 14, in which the footing course is stepped out on all sides of the load to be carried, may cause cross-stresses in the footings, two sets of steel rods placed at right angles to each other should be used.

39. Reinforced concrete may be employed to bridge over spaces between piles or other non-continuous supports. Thus, in Fig. 17, where the two rows of piles *A* and *B* are spaced far

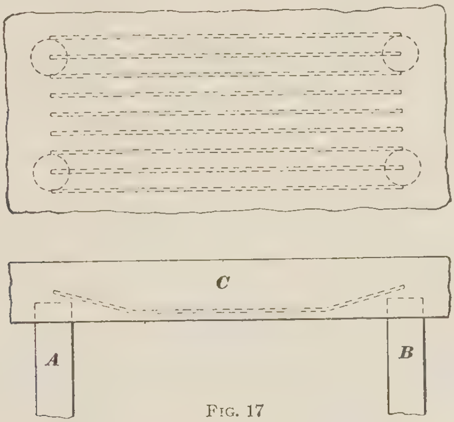


Fig. 17

apart, and the soil between them is so soft as to afford little or no support, a plain-concrete cap *C* would be inadequate to carry the distributed load, but the cap may be made amply strong by reinforcing it with steel rods, as shown. In this case, the beam is to be treated as a beam supported at both ends, and its bending moment calculated accordingly. Its resisting moment may be found by formula **1**, Art. **38**.

Example 1.—Let Fig. 16 represent a section of a wall *A* resting on a single footing course *B* of reinforced concrete, the dimensions being as marked, and the weight 20,000 pounds per foot of length. The steel reinforcement consists of three plain rods each $\frac{3}{4}$ inch square, evenly spaced in the 1-foot section, at a distance of 15 inches from the top of the course. To find the moment of resistance of the footing and to determine whether the beam is strong enough, using the constants recommended by the Joint Committee.

SOLUTION.—To apply formula **1**, Art. **38**, $A = (\frac{3}{4})^2 \times 3 = 1.6875$; $d = 15$; $b = 12$. Then, by formula **3**, Art. **38**, $p = \frac{A}{b\,d} = \frac{1.6875}{12 \times 15} = .0094$. Referring to columns 1 and 2, Table II, *Reinforced Concrete*, the corresponding value of R is found as 114.67. Substituting known values in formula **1**, Art. **38**,

$$M = 114.67 \times 12 \times 15^2 = 309{,}600 \text{ in.-lb.} \quad \text{Ans.}$$

By the formula of Art. **37**, the bending moment is

$$M = \frac{20{,}000 \times 48}{4} = 240{,}000 \text{ in.-lb.}$$

The moment of resistance exceeds the bending moment, and the beam is, therefore, strong enough. Ans.

EXAMPLE 2.—Assuming the conditions of the preceding example, let the weight of the wall be 34,000 pounds per linear foot and the steel reinforcement be placed 20 in. from the top of the footing. Find the area of steel that would be just sufficient to carry the load.

SOLUTION.—Apply formula **2**, Art. **38**. Solving for R, it is found that

$$R = \frac{W\,n}{4\,b\,d^2}$$

To apply this formula, $W = 34{,}000$; $n = 48$; $b = 12$; and $d = 20$. Substituting these values,

$$R = \frac{34{,}000 \times 48}{4 \times 12 \times 20^2} = 85$$

Referring to Table II, *Reinforced Concrete*, it is found that for the constants recommended by the Joint Committee, the value of P corresponding to $R = 85$ is .0060. Applying, now, formula **3**, Art. **38**, the required area is

$$A = p\,b\,d = .006 \times 12 \times 20 = 1.44 \text{ sq. in.} \quad \text{Ans.}$$

ECCENTRIC LOADING OF FOUNDATIONS

40. When the resultant pressure acting on a foundation does not pass through the center of gravity of the foundation, the latter is said to be **eccentrically loaded.** In the absence of external lateral forces, the center of pressure of a foundation lies in a vertical line passing through the center of gravity of the structure. If this line does not pass through the center of gravity of the foundation, the latter is eccentrically loaded. One of the most frequent examples of eccentrically loaded foundations is that of city buildings, where the wall of one building stands directly against that of its neighbor, as illustrated in Fig. 18. The footing courses can be stepped off on one side only, with the result that while

the center of pressure falls at a, the center of gravity of the foundation is at b. This causes an uneven distribution of pressure over the subfoundation, and is likely to produce uneven settlement and consequent cracking of the structure. Where the subfoundation is absolutely unyielding, as solid rock, eccentric loading may generally be disregarded, unless it is sufficient to crush the material. As, however, masonry with mortar joints is likely to be somewhat compressible, it is better to avoid eccentric loading even in such cases. In the case of all structures resting on compressible subfoundations, eccentric loading should be avoided as much as possible, and, where it is not practicable to avoid it, it should be properly provided for.

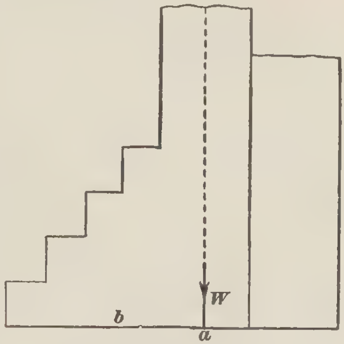


FIG. 18

MATERIALS USED FOR FOUNDATIONS

41. The materials commonly employed for foundations are masonry and timber. Steel and iron are occasionally used alone, but only under exceptional circumstances; steel rods embedded in concrete, however, are very extensively employed, reinforced concrete being now one of the materials of construction most commonly used for nearly all kinds of structures. The successful designing of foundations requires a knowledge of the properties of these materials and of the conditions and forms of construction to which they are adapted. The subject of masonry, including both plain and reinforced concrete, is fully treated in other Sections of this Course, and here it will be sufficient to make only a few remarks relating to the use of the various kinds of masonry for foundation work.

MASONRY

STONE AND BRICK MASONRY

42. Although concrete has to a great extent superseded stone and brick for foundation work, the two latter materials are still used to a very great extent. Of the several kinds of stone masonry described in *Stone and Brick Masonry*, the only ones that are commonly used for foundations are squared-stone and rubble masonry.

43. Kind and Quality of Stone Used.—The stones usually employed in masonry construction are granite, limestone, sandstone, trap rock, and occasionally massive slate. Any of these may vary in quality so widely that no stone without an established reputation for strength and durability should be used in important structures except after careful investigation. In the older settled sections of the country, the quality of stones from certain geological strata and from certain quarries has been established by long experience, and these may be used with confidence. In new countries or localities, the engineer must, in the absence of experience, determine the character of any stone by proper tests. Numerous physical and chemical tests have been proposed or devised for determining the quality of building stone, but they are often unnecessary. In many cases, strata of a rock may be observed along streams or the sides of hills, where it has been exposed for centuries to destructive climatic influences; the present condition of these strata is the surest evidence of the quality of the rock. If the exposed parts stand out boldly, with sharp angles and projections and no cracks, no better evidence of durability could be wanted; and it is also found that under such conditions the strength of the rock is usually high. Such evidence is of far greater practical value than any chemical or physical tests that can be applied. Furthermore, no physical or chemical tests should be accepted as conclusive of the value of the rock if in such natural exposures it appears cracked and disintegrated.

with distinctly rounded angles and projections, or has receded by decay under or around projecting strata of greater durability. It is true that some such rock, if quarried and properly dried or seasoned, may prove fairly durable in structures where its dryness may be maintained or where it will be more or less protected from climatic influences; but, as a rule, experience alone can safely establish such qualities, particularly in foundation work, where the stone, being exposed to moisture and alternations of heat and cold, is likely to develop any weaknesses that would appear from its natural geological exposure.

TABLE IV

Kind of Stone or Brick	Ultimate Crushing Strength		Ultimate Tensile Strength
	Pounds per Square Inch	Tons per Square Foot	Pounds per Square Inch
Limestone	6,000 to 18,000	430 to 1,300	140 to 2,500
Marble	7,000 to 20,000	500 to 1,440	150 to 2,800
Sandstone, hard	6,000 to 15,000	430 to 1,080	600 to 2,300
Sandstone, soft	4,000 to 10,000	290 to 720	150 to 1,200
Granite	10,000 to 20,000	720 to 1,440	900 to 2,700
Trap rock	15,000 to 24,000	1,080 to 1,730	
Brick, ordinary building .	600 to 7,500	43 to 540	250 to 2,700
Brick, pressed	800 to 15,000	58 to 1,080	500 to 2,000
Brick, vitrified paving . .	8,000 to 18,000	58 to 1,300	1,500 to 3,000

44. Strength of Stone and Brick Masonry.—The engineer is most interested in the compressive resistance of stone or brick, and does not often have occasion to consider the tensile strength. The former has been determined in a large number of instances, but data on the latter are few and unsatisfactory. The resistance to crushing of the same kind of stone varies within wide limits. An idea of these variations may be obtained from Table IV. In the last column of this table is given the approximate tensile strength of the materials, according to the best available authorities. Some of the values for crushing strength have already been given in Table I. These values are given here for genera'

information, not as the values that would be likely to be used in any actual case.

45. Cutting and Dressing Stone.—For ordinary foundation work, where the stones are laid in good Portland-cement mortar, very little cutting and dressing is required, and money expended to secure close beds or joints, or good faces, is practically wasted. Even where squared-stone masonry is used, the stone, if it comes from the quarry in reasonably good shape, needs very little dressing. It can be broken by the hammer into roughly approximate form and dimensions, and it may be necessary to use the point chisel to remove objectionable projections. Proper care to bed the masses in and to thoroughly fill the joints and all

TABLE V

CRUSHING STRENGTH OF MASONRY

Kind of Masonry	Crushing Strength Tons per Square Foot
Rubble, dry	3 to 10
Rubble in cement mortar	10 to 15
Squared stone in cement mortar	15 to 20
Ashlar limestone in cement mortar	20 to 25
Ashlar granite in cement mortar	25 to 30
Brick in lime mortar	3 to 5
Brick in cement mortar	5 to 12

voids with good mortar will insure the requisite strength and stabililty at a far lower cost than careful and expensive dressing. This is still more true of rubble masonry, where no tool other than the hammer need be used, and that but sparingly.

46. Safe Loads on Masonry.—There is probably no definite or even approximately close ratio between the crushing resistance of the stone or brick of which masonry is built and that of the masonry itself. Nor is it possible, with any facilities that have been or are likely to be available, to determine the crushing resistance of large masses of

masonry. In a few instances, masonry has failed under excessive loading, but in even these few cases there are not sufficiently complete data to determine the crushing resistance with any degree of accuracy. Such experience indicates that masonry of the classes mentioned in Table V may be expected to carry safely the loads stated. The lower value applies to good and the higher to the best quality of masonry of its class.

In the great majority of cases, the dimensions of a masonry structure are determined by considerations other than capacity to carry load, and the actual load imposed is so far below the ultimate strength of the masonry that this strength is not an important matter.

CONCRETE

47. Plain and Reinforced Concrete.—The subject of concrete is fully treated in *Plain Concrete* and *Reinforced Concrete*. As there stated, concrete made with Portland cement is far preferable to that made with natural cement, especially now that, owing to the development of the Portland-cement industry, this material can be obtained at comparatively low prices. In general, Portland cement is more economical than natural cement for many kinds of work; that is, for the same strength of concrete, one dollar's worth of Portland cement will make a larger volume of concrete than will one dollar's worth of natural cement.

In localities where, for any reason, natural cements are much cheaper than Portland cements, natural-cement concrete may be the more economical to use, and for many purposes its strength will be found sufficient.

For most engineering works, mortars of natural cement and sand should be in the proportion of one part of cement to two parts of sand, and concrete made of the same material should usually consist of one part of cement, two parts of sand, and four to five parts of stone. Such concrete should, at the age of 6 months, have a crushing strength of 900 to 1,000 pounds per square inch, and may be safely loaded with 8 to 10 tons per square foot.

48. Stone and Brick Masonry Combined With Concrete.—Where it is thought necessary that the outer or exposed surface of foundation masonry shall be of stone, in order to more effectively resist floating ice, heavy drift, or concussion and abrasion from passing vessels, the face of the structure may be built of any desired class of stone or brick masonry, and the interior filled with concrete, which is put in place as the masonry courses are completed. If the work is

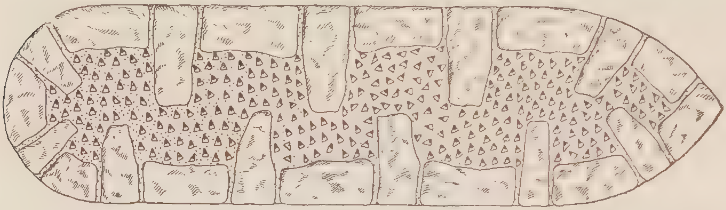


FIG. 19

properly done, this method of construction is effective and economical. Fig. 19 illustrates, in plan, this method of construction.

TIMBER

KINDS AND PROPERTIES OF STRUCTURAL TIMBER

49. Kinds of Wood Commonly Used.—In a well-timbered country, wood is the most convenient and economical structural material. The main objections to it are that it is comparatively short-lived, and that it is easily destroyed by fire. There is a wide difference in the strength and durability, as well as the abundance and availability in different localities, of the different species of wood. Some of them, when exposed to the influences of the weather, will decay in a very short time, while others are fairly durable, even under the most unfavorable conditions.

The woods that have been most extensively employed for structural work are the following:

1. The *white oak family*, which includes common white oak, post oak, and chinkapin oak. These woods possess a

high degree of strength and hardness, and are among the most durable. Other species of oak are comparatively hard and strong, but lack durability.

2. The *pines* are important structural woods. In the northern part of the United States, white pine has long been a standard building material. It is strong, but lighter than most woods; it is soft and easily worked. The long-leaf yellow pine of the American Atlantic and Gulf States is one of the strongest, hardest, and most durable of woods, comparing favorably in these respects with white oak. On the Pacific Coast, the Oregon *red fir* is one of the best woods for structural work.

There are many species of pine that are almost equal in strength to those named, but they are short-lived. In the Southern States, loblolly pine and bastard pine are abundant, and when sent to market it requires a good knowledge of the characteristics of each to distinguish the timber from long-leaf yellow pine. In fact, a large part of the timber and lumber now on the market purporting to be long-leaf yellow pine is really loblolly or some other inferior species of pine. The engineer will require expert knowledge to be able to distinguish and reject the inferior kinds.

3. *Cypress* is a very durable wood found mostly in the swamps of the southern American states. While possessing sufficient strength, its liability to split makes it unsuitable for many structural purposes.

There are a number of other species of wood whose combined qualities of strength and durability recommend them to the engineer, but they are not sufficiently abundant to make them available for extensive structural work.

50. When, for any reason, the question of durability need not be considered, as for temporary constructions, or for foundations or parts of structures constantly submerged in water, or buried in damp clay to such a depth as to exclude the air, there are many other species of wood that possess sufficient strength and are found in more or less abundance. Among these are all the species of oak, most of the pines,

hemlock, spruce, elm, ash, hickory, and others. For submerged foundation work, any of these may be utilized safely.

51. Strength and Durability of Timber.—Table VI gives the tensile and crushing strength of the species of

TABLE VI

STRENGTH AND DURABILITY OF DIFFERENT SPECIES OF WOOD

Species of Timber	Ultimate Tensile Strength Pounds per Square Inch Parallel to Fiber	Ultimate Crushing Strength Pounds per Square Inch		Relative Durability on Scale of 1 to 10
		Parallel to Fiber	Across Fiber	
Ash, white	10,800	7,200	1,300	6
Birch, black	7,000	8,000	1,300	2
Cedar, white	6,300	5,200	500	8
Chestnut	13,000	5,300	900	6
Cypress, bald	7,900	6,000	500	9
Elm, white	10,300	6,500	1,300	4
Gum, sweet	9,500	7,100		3
Hemlock	6,500	5,300	600	2
Hickory, shagbark	16,000	9,500	2,000	5
Locust, black	18,000	9,800	1,900	10
Maple, white	10,000	6,800	1,300	2
Oak, white	13,100	8,500	1,700	8
Oak, red	11,400	7,200	1,600	5
Pine, white	7,900	5,400	600	7
Pine, long-leaf yellow	10,900	7,000	1,300	9
Pine, loblolly	10,100	6,500	1,200	3
Pine, short-leaf	9,200	5,900	1,000	3
Spruce, black	10,000	5,700	700	6
Spruce, Douglass	7,900	5,700		8

wood commonly employed in structural work. The values given relate to sound, well-seasoned wood. In the last column, it has been attempted to designate on a scale of

1 to 10 the relative durability of the species named, 1 representing the least durable, and 10 the most durable woods.

It must be noted, however, that the figures in this table are only approximately correct. The conditions of age, soil, climate, etc., under which the wood has grown, greatly affect both its strength and its durability. The values given in the table may be safely used with a factor of safety of about 10.

SEASONING AND PRESERVATION OF TIMBER

52. Causes of Decay.—As is generally known, the decay of wood is caused by the growth in it of fungi, many of them microscopic. These fungi require moisture and air for their development and growth. In the absence of either air or moisture, they can neither propagate nor grow. When wood is completely submerged in water, access of sufficient air to support these fungi growths is prevented, and almost any kind of wood, however short its life in open air, will remain sound for long periods of time. For this reason, species of wood that are very short-lived under ordinary conditions will prove sufficiently durable in foundations that are either always submerged in water or buried to considerable depths in soil saturated with water. Any kind of wood, therefore, that possesses the requisite strength may be safely used in submerged foundation work, provided that it is not attacked and destroyed by the Teredo or other wood-boring worms. If such woods, after thorough seasoning, could be kept perfectly free from moisture, they would prove durable even when exposed to the air. This is true of all woods, whether naturally durable or not. On the contrary, the durability of any wood will be greatly abridged if the wood is full of moisture when placed in structures in situations where the moisture will be retained, and access of air freely permitted.

53. Seasoning.—The thorough drying of wood intended for structural work is important. The process of drying is called **seasoning**, and it may be either a natural or an

artificial process. Natural seasoning consists in exposing the wood to sun and air, while protecting it from rain. In artificial seasoning, the wood is subjected to artificial heat, with free access of air. The process is usually conducted in enclosed spaces called **kilns**, and the name **kiln-dried** is often applied to wood thus seasoned. The operation is greatly expedited by promoting the free circulation of air to carry away the moisture as rapidly as it is evaporated, and for this purpose power fans are often employed to keep up a constant air circulation. The process is still further hastened by first heating the air before it is forced into the kilns. Natural seasoning may require from 3 months to 1 year, depending on the conditions, while artificial seasoning may require but a few weeks. For most foundation work, where the timbers will be either submerged or subjected to constant moisture, seasoning is not necessary.

54. Chemical Preservation of Timber.—A large number of processes for preserving wood from decay and from the attacks of worms have been proposed and used more or less. They nearly all consist in filling the pores of the wood with some antiseptic that is supposed not only to destroy and prevent the growth of fungi, but to exclude moisture. Some of the chemicals used are soluble in water. Such chemicals will usually leach out when the timber becomes wet, and after a few years disappear, leaving the timber in the same condition as if it had never been treated. A few of the various processes will be very briefly described.

55. Vulcanizing.—By this process, the wood is heated in a close cylinder with air at a pressure of from 100 to 175 pounds per square inch, and a temperature of from 300° to 500° F. It is claimed that the action of the heated air evaporates all the water in the wood and coagulates the sap; that the high degree of heat sterilizes the wood and produces such changes as to render it antiseptic; and that the results are, in every other respect, very satisfactory. Ties have been treated in this way on a large scale at a cost of about 25 cents per tie.

56. Creosoting is one of the most efficient and satisfactory wood-preserving processes yet discovered. It consists in forcing into the wood the dead oil of coal tar commonly known as **creosote oil.** To properly apply the process requires an expensive plant, and increases the cost of the timber very materially. The timber to be treated is placed in a large iron vessel or cylinder, which is then closed air-tight. The cylinder is filled with live steam at a pressure of at least 50 pounds per square inch (very soft wood may require as high a pressure as 144 pounds per square inch) to heat the timber and extract the sap from it. After about 6 hours' exposure, the steam is shut off, the condensed water, sap, etc. removed from the cylinder, which is then connected with an air pump, and the air exhausted until the barometer pressure does not exceed 12 inches. The partial vacuum is maintained for a period of from 5 to 8 hours, the object being to extract the sap and moisture from the wood as completely as possible. While the partial vacuum is still maintained, the dead oil, at a temperature of about 220° F., is admitted to the cylinder until the wood is entirely covered by it. The air pump is then disconnected and a force pump connected with the cylinder, by which a pressure of at least 40 pounds per square inch is produced and maintained until the wood has absorbed the desired quantity of oil. The quantity of oil injected into the wood varies with the purpose for which the timber is to be used. For railroad cross-ties, telegraph poles, railroad trestles, etc., 10 to 12 pounds of oil per cubic foot of timber is considered sufficient; while, where the object is the protection of the wood from sea worms, from 16 to 25 pounds of oil per cubic foot should be forced into the wood. No definite figures can be given, since some woods are more porous than others and require a larger quantity of oil to properly saturate them.

57. The **creo-resinate process** consists in impregnating the wood with a compound of sixty parts of creosote, thirty-eight of rosin, and two of formaldehyde. Creo-resinate is applied in the same manner as plain creosote.

This process is the most recent, and is considered to be effective, both from an economic and from a sanitary point of view, though each of the other processes, when properly applied, will prevent decay and thus lengthen the natural life of the wood. All processes greatly diminish expansion and contraction, and also render the wood practically impermeable.

58. The **zinc-tannin process**, which is also known as the **Wellhouse process**, after the inventor, consists in successively impregnating the wood with two solutions; as soon as these two solutions become mixed within the pores of the wood, a chemical reaction takes place, which turns them into a leathery substance that is insoluble in water and cannot, therefore, be readily washed out when the timber becomes wet. The chemicals employed in this process are a solution of chloride of zinc and glue, which is first forced into the wood by pressure, and a solution of tannin, which is forced into the wood after the first solution. It has been claimed that such a process, which may be applied to a cheap soft wood tie as readily as to the best oak tie, will treble the life of a soft wood tie, while it costs about 11 to 13 cents per tie. It is even claimed that the spikes in wood treated in this manner have a greater holding power than in the plain wood.

59. Other Processes.—Several other processes are employed for treating timber. One of the most common of these is **kyanizing**; this consists merely in soaking the timber in a tank containing a solution of corrosive sublimate (bichloride of mercury). The proportions used are 1 pound of sublimate to from 10 to 15 gallons of water. The process, although it has been extensively used, is objectionable in many respects, and is not effective for long periods.

Another process, known as **burnettizing**, consists in forcing into the wood, under pressure, a solution of chloride of zinc. In this process, the sap is previously removed from the timber by the method explained in Art. **56** for creosoting.

QUALITY OF TIMBER

60. Timber is likely to vary more widely in quality, and to require more careful inspection, than any other material with which the engineer has to deal. Growing trees are subject to injuries and disease that cannot be ascertained until the trees are cut into lumber. The best and strongest timber comes from thrifty live trees that have reached but not passed the age of maturity. Such trees consist of two well-defined portions, **heart wood** and **sap wood**, the latter being the outer and less matured part. Sap wood is generally not so strong and decays much more quickly than heart wood. The best of timber for structural purposes, where durability is to be considered, is free from sap wood. If, however, the timber is to be constantly submerged in water. or if it is to be treated by creosoting, the sap wood is acceptable. Structural timbers should be straight-grained, free from rotten or decayed parts, and from incipient decay, commonly called *dotiness;* from large knots, particularly if they are unsound; from cracks, whether those known as wind shakes or caused by improper seasoning; and, in short, from any imperfections that may impair their strength and durability.

STEEL AND IRON

61. Foundations are not often constructed wholly or even mainly of steel or iron, but special conditions have led to such constructions in not a few instances. Where great strength must be developed within limited spaces, or great weight must be transferred from one point to another, steel is the only adequate material. Iron and steel are used in comparatively small quantities, as for bolts and clamps in timber and masonry work; steel rods are important elements in the construction of reinforced concrete. In foundation work, where the loads are wholly static or quiescent, fiber stresses of from 14,000 to 16,000 pounds per square inch are permissible for plain steel; and in reinforced-concrete construction, where steel rods having an elastic limit of 45,000

pounds per square inch or over are used, they may be safely loaded up to 15,000 pounds per square inch of section.

62. The chief objection to the use of wrought iron and steel in foundation work is their liability to oxidize or corrode in the presence of air and moisture. Even where the metal is permanently submerged in water, the oxygen of the air contained in the water may be sufficient to cause serious oxidation, unless the metal is protected in some way. The important fact seems well established, though the reason for it is not fully understood, that iron or steel embedded in concrete, whether dry or wet, does not oxidize, and will remain for at least very long periods of time free from rust or deterioration. Combinations of steel and concrete are especially applicable to some foundation work, and, when constructed with proper care, such work may confidently be expected to prove durable and satisfactory. All steel and wrought iron used in foundations should be encased in concrete wherever possible. Cast iron is comparatively free from oxidation, even when exposed to moist air under the most unfavorable conditions.

FOUNDATIONS

(PART 2)

FOUNDATIONS ON LAND

CLASSIFICATION OF FOUNDATIONS

1. Bases of Classification.—While foundations proper may be made of a considerable variety of materials or of combinations of materials, and may differ widely in their forms and in the methods of constructing them, they may be classified with reference to either the kind of material used, or the method of construction.

2. Classification Based on Kind of Material Used. With reference to the materials employed, foundations are divided into the following classes: (1) *masonry foundations*, which consist wholly or largely of some class of masonry, such as stone, concrete, or brick; (2) *timber foundations*, which are constructed of timber, usually sawed or hewn into rectangular form (though round timber is sometimes used), framed or otherwise connected to form a solid structure or mass; (3) *pile foundations*, in which piles of either wood, metal, or concrete form the essential part of the foundation; and (4) *steel foundations*, which are made of steel.

3. Classification Based on Method of Construction. With regard to methods of construction, foundations are thus classified: (1) *dry foundations*, in which, as the name implies, no water is encountered, and no special provision for it must be made: foundations of this kind are comparatively rare; (2) *submerged foundations*, which are partly or

wholly under water; (3) *wet foundations*, into which water flows or percolates, so that bailing or pumping is necessary; (4) *crib foundations*, which are wholly or partly composed of timber cribs or rafts; (5) *coffer-dam foundations*, in which a coffer dam is used during construction; and (6) *pneumatic foundations*, in the construction of which compressed air is used to assist the sinking of the foundation into place.

PILE FOUNDATIONS

PILES

4. Kinds of Piles.—A **pile** is a column or beam of wood, metal, or other material, driven into the earth to support or protect a structure.

Wooden piles are ordinarily the trunks of trees, freed of branches and large knots. For special purposes, however, piles sawed or hewn into rectangular sections are frequently used.

Iron and steel piles of various forms and sections are sometimes employed. If of cast iron, they are usually hollow cast tubes or columns; if of steel, they consist of single rolled shapes or of two or more such shapes riveted together to form a single member.

Concrete piles consist either of concrete reinforced with steel rods, or of plain concrete.

5. Whatever may be their form or material, piles are often classified with reference to the special purpose they are intended to serve.

Bearing piles are those used to support vertical forces or loads; to this class belong the great majority of piles used in engineering construction.

Protection piles are piles driven singly or in groups or rows to protect or shield a structure from external injury.

Anchor piles, or **mooring piles,** are used singly or in groups for holding or anchoring vessels and other floating structures.

Sheet piles consist of rows of piles, driven together as closely as possible, to enclose a foundation or other space; they are usually of rectangular section and are often tongued and grooved to insure close contact and to prevent the passage between them of water or mud.

Some special varieties of piles have received distinctive names on account of their forms or the manner of putting them in place. Among such may be mentioned *screw piles*, *jet piles*, and *pneumatic piles*. These will be described more fully in subsequent pages.

6. Use of Piles for Foundations.—Pile foundations are resorted to when the surface soil at the site of a structure is unsuitable for a subfoundation and suitable material exists only at such a distance below the surface that it is impracticable to reach and utilize it by the ordinary methods of excavation. Even if fairly satisfactory material is available, piles serve to distribute the load through a greater vertical depth than would be otherwise practicable.

THE DRIVING AND SINKING OF PILES

7. Pile Drivers.—Piles of small size to be driven to comparatively short depths may be forced into position by sledges or mauls in the hands of workmen. Larger piles are driven by special machines, called **pile drivers.** These drivers are of two kinds; namely, *drop-hammer pile drivers*, and *steam-hammer pile drivers*.

8. Drop-Hammer Pile Drivers.—Fig. 1 illustrates a very common type of drop-hammer driver. It consists essentially of a heavy weight a, called the **hammer**, and of motive power and mechanism for raising the hammer and allowing it to fall on the top of the pile. The hammer is usually of cast iron, of the form shown at (a), and weighs from 1,000 to 3,000 pounds. It is cast with vertical grooves or recesses a', a', which partly envelop and engage with the upright guides b, b, called the **leads.** These are two timbers supported on a horizontal platform, and serve to guide the

hammer in its vertical motion. The leads are braced and stiffened by inclined struts c, c. A line r of rope or wire cable has one of its ends attached to the hammer, from which it passes over a sheave at the top of the leads and thence to the drum d of a hoisting engine. This drum is connected to the engine through a friction clutch, by which it may be quickly thrown into and out of gear. Another drum d_1 similarly connected to the engine, operates a second cable r_1 passing over a sheave at the top of the leads and used for hoisting the pile into place.

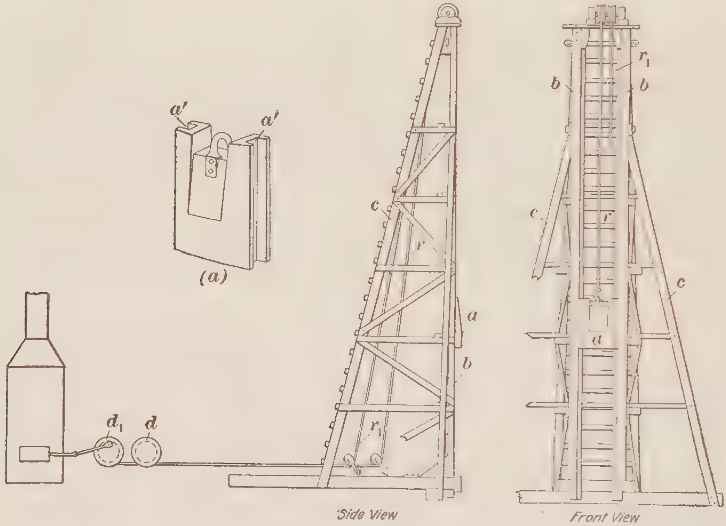


FIG. 1

In operation, the hammer is raised out of the way, and the pile is hoisted and placed upright between the leads, with its foot on the ground where it is to be driven, and secured in position by appropriate devices. The hammer is gradually lowered on the head of the pile, to force the latter into the ground to a slight depth, dependent on the softness of the soil. The hammer is then raised to any desired height, the engine stopped, and the friction clutch released, when the hammer drops on the head of the pile. In doing so, it draws the cable with it, reversing the motion of the drum,

which is partly controlled by a brake that prevents it from continuing to revolve after the hammer has struck the top of the pile. This operation is repeated until the pile is driven to the desired depth.

9. The method just described of attaching the hoisting line permanently to the hammer is the one now most commonly used, and is considered the most satisfactory. Some machines are, however, still equipped with an automatic catch, called a "nipper," between the end of the line and the hammer. This device is illustrated in Fig. 2. The nippers proper *b*, *b* are attached to a cross-beam *a* that has recesses at its ends and slides up and down on the leads. The lead line is attached to an iron piece having an eye at its top, as shown. To the top of the hammer is attached a clevis having a wedge-shaped cross-piece *d* between the sides. When the nippers are lowered, they slide over this wedge-shaped piece and engage with it; and when the line is hoisted, it carries the hammer with it. At the top of the leads are placed tripping blocks, which press the curved arms together, thus releasing the hammer. The line is then lowered, the nippers again engage the hammer, and the operation is repeated. This device is seldom used now except with small drivers operated by horses.

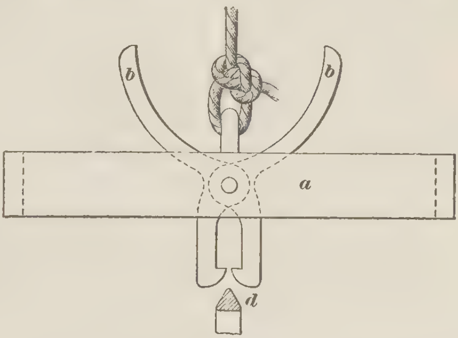


Fig. 2

10. Steam-Hammer Pile Drivers.—A steam-hammer pile driver differs from a drop-hammer machine in that the hammer is operated by steam, in the same general way as the steam hammers used for forging iron and steel. The mechanism consists of a vertical steam cylinder with a piston having a stroke of about 3 feet, the lower end of the

piston rod carrying a heavy hammer. Fig. 3 illustrates the construction of the steam hammer, though the machines in actual use are more elaborate: *a* is the steam cylinder, *b* the piston rod, and *c c* the hammer, *d*, *d* are guides the upper ends of which are attached to the steam cylinder, and the lower ends to the anvil block or plate *e*. These rods pass through holes in both sides of the hammer, thus guiding its motion. The block *e* has a conical hole through its center, as shown by dotted lines, and the hammer has a projection *f* that passes freely through this opening.

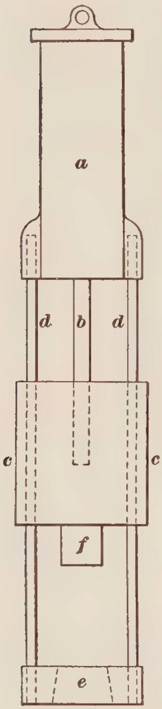


Fig. 3

Steam, conveyed to the cylinder by a flexible pipe or hose, is admitted below the piston, forcing it and the attached hammer upwards. At the end of the upward stroke, the steam is automatically cut off, the exhaust port is opened, and the hammer and piston fall by their own weight to their original position; steam is again automatically admitted, and the operation repeated. The whole mechanism is placed between and guided by the leads, and is suspended by the line that, in the drop-hammer machine, carries the hammer. When the pile is in place, the whole apparatus is lowered on its top, which is usually dressed to fit approximately the conical hole in the anvil block. Steam is then turned on and the hammer begins to work, striking rapid blows and following the pile as it descends.

11. Comparison of the Two Kinds of Pile Drivers. There has been much discussion as to the comparative merits of these two types of pile drivers. The truth is that each is most effective under conditions that favor its use. In the drop-hammer machine, the hammer may be allowed to fall any distance from 1 to 30 or more feet; while in the steam hammer the fall is limited to the stroke of the piston, usually

from $2\frac{1}{2}$ to $3\frac{1}{2}$ feet. As with other falling bodies, the energy developed by the drop of the hammer is measured by the product of the weight and the distance through which it falls. With hammers of equal weight, it is obvious that the drop hammer, with its much greater possible range of fall, is capable of delivering the more energetic blows, and is therefore more effective where the material to be penetrated is very hard. On the other hand, the steam hammer delivers its blows much more rapidly than the drop hammer, and, if the material through which the piles are to be driven is comparatively soft, so that the energy of the blow is adequate to force the pile through it, more rapid progress may be made than with the drop hammer. Thus, if a steam hammer weighing 2,000 pounds and falling $3\frac{1}{2}$ feet strikes eighty blows per minute, the work done will be

$$2{,}000 \times 3\tfrac{1}{2} \times 80 = 560{,}000 \text{ foot-pounds per minute;}$$

while a drop hammer of the same weight falling 25 feet may strike ten blows per minute, and the mechanical work developed will be

$$2{,}000 \times 25 \times 10 = 500{,}000 \text{ foot-pounds per minute.}$$

In practice, the steam hammer is made from 25 to 50 per cent. heavier than the drop hammer, and this partly compensates for the smaller fall of the former. The greater energy of the blow of the drop hammer, where it is not required to force the pile through hard material, is disadvantageous, because it is much more likely to split or "broom" the head of the pile. On the other hand, the first cost of the steam-hammer outfit is much greater than that of the drop hammer, and its more complicated mechanism makes it more liable to break down and more expensive to keep in repair. The drop-hammer machine is preferred by almost all contractors.

12. Inclined Piles.—Piles are usually driven vertical. When, in exceptional circumstances, it is necessary to have them inclined, this is easily done by simply inclining the leads of the pile driver.

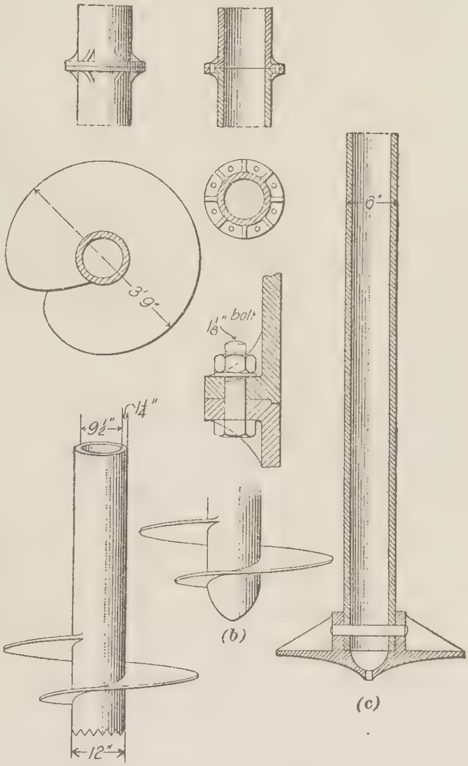
13. Location of Pile Drivers.—For building and other similar work, the pile-driving mechanism is installed on the

ground, near the place where the piles are to be driven. When piles are to be driven in water, the pile driver is placed on a flatboat or scow, so that it will be readily movable from point to point. Pile drivers for work on operating railroads are placed on flat cars in such a manner that they can be revolved in a horizontal plane, thus allowing the leads to be swung through a considerable arc from side to side, in order to drive the piles in any position required. Drivers are also constructed so that the leads can be inclined to drive the outer or batter piles at the proper angle with the vertical.

14. Sinking Piles by Water Jet.—Where piles are to be used in sand or comparatively loose soil, they may be sunk into position by the use of a jet of water. In this method, the lower end of the pile is fitted with a nozzle discharging downwards. This nozzle is connected by hose or pipe to a force pump. When the pile is placed in position, the pump is started and a strong jet of water is discharged against the sand or other material immediately below the end of the pile. The result is to soften and cut out the material, allowing the pile to settle either by its own weight, or by additional weight placed on its top, or, sometimes, by comparatively light blows of a pile driver, until the desired depth is reached. When the operation is completed and the current of water discontinued, the surrounding material settles into close contact with the pile, giving it the necessary stability and bearing power. Under favorable conditions, piles may be driven by this method more rapidly and cheaply than with the ordinary pile driver. This is particularly true in dry sand, which offers great resistance to piles forced into it by blows from a hammer.

15. Screw Piles.—Iron or steel piles, and occasionally wooden piles, sometimes have their points provided with a horizontal spiral flange or screw, by the aid of which they are forced into sand or soft soil. Usually, one or two turns of the flange or thread are attached to the lower end of the pile, as shown in Fig. 4 (*a*) and (*b*). When such a pile is

placed in position and revolved in the proper direction, the spiral blade or screw engages with the earth, forcing the pile downwards in much the same way as an ordinary wood screw is forced into soft wood. The spiral flanges vary in width, but are ordinarily from 2 to 5 feet in diameter. When once started, screw piles may be sunk pretty rapidly. The power required to turn the pile will vary with the hardness of the material to be penetrated. In soft material, the driving is accomplished by men working at levers, similar to the levers of a capstan, attached to the pile. In harder material, mechanical power is used, which is applied to the pile through suitable devices. The shaft may be made of sections of steel or cast iron connected by screws or flanges, as shown in the figure.



(a) FIG. 4

One advantage of this form of pile is that the wide flange greatly increases the bearing area of the pile in soft material, where the ordinary pile might not be effective. Screw piles are frequently used in preparing foundations under water or in soft boggy locations.

CONCRETE PILES

16. Concrete piles are used to a considerable extent in foundation work. They possess the great advantage that they do not decay like wood nor corrode like steel piles. Two types will be described here.

A tube or shell of thin steel, shaped like the intended pile, is constructed so that it is partly collapsible along one or more longitudinal lines or points. This shell, which is called a **pilot pile,** is driven into the ground in much the same manner as the ordinary wooden pile. When driven, it is partly collapsed and withdrawn, and the hole made by it is filled with concrete. Where the ground is soft and the walls will not maintain their form after the pilot pile is withdrawn, a thin casing of metal or other material fitted over the pilot pile is driven with it; and, when the latter is withdrawn, the casing preserves the walls of the hole until it is filled with concrete. This kind of concrete pile has been very successfully used for the foundations of heavy buildings.

In another form of concrete pile, the pile is made of concrete in molds of the desired shape and dimensions, and allowed to set or harden. It is then driven by the water-jet process or by careful driving with a pile driver. These piles are usually reinforced with steel rods or bars, and are thus made sufficiently strong to be readily handled and driven. They are also usually provided with iron or steel shoes, to prevent injury to their points and to facilitate driving. If hammer-driven, their tops are protected with an elastic cushion of wood or other material.

SUPPORTING POWER OF HAMMER-DRIVEN PILES

17. Theoretical Formula.—Many efforts have been made by engineers to derive a satisfactory formula for the bearing capacity of piles. These formulas are usually derived from a theoretical analysis of all the conditions involved. The resistance of the ground is determined by the penetration of the pile during the last few blows of the pile driver.

Experience has proved that it is practically impossible to derive a purely rational formula, on account of the uncertainty of the conditions. It has been found that the only reliable practical formulas are those that, although having a theoretical basis, have been modified according to the results of experience.

Let R = resistance or bearing capacity of a pile;
s = set of pile, or distance that pile is driven during last blow;
w = weight of pile hammer;
h = fall of hammer during last blow.

Then, according to the principles of mechanics,

$$R s = w h;$$

and, therefore,

$$R = w h \div s$$

This is the basis of all theoretical formulas. In this formula, R and w are in the same units, such as pounds, tons, kilograms; also, h must be in the same units as s, whether feet, inches, centimeters, etc. If the set s of the pile under the last blow is expressed in inches, and the fall h of the hammer in feet, the formula becomes

$$R = 12\, w h \div s$$

If a factor of safety of 6 is adopted, then

$$R = 2\, w h \div s$$

18. Engineering News Empirical Formula.—On account of the extra resistance caused by the settling of the earth around the pile between successive blows, the divisor in the last formula of the preceding article has been arbitrarily increased from s to $s + 1$ for drop-hammer drivers, and to $s + .1$ for steam-hammer drivers, and the following practical formulas have been proposed:

For drop-hammer pile drivers,

$$R = \frac{2\, w h}{s + 1} \qquad (1)$$

For steam-hammer pile drivers,

$$R = \frac{2\, w h}{s + .1} \qquad (2)$$

Formula 1 is called the **Engineering News formula,** because it was first published by that engineering journal.

It has been very extensively adopted, as experience has proved that it is as reliable as any formula can justifiably claim to be. The uncertainties of pile driving are so great that it is useless to attempt to use a more accurate formula.

EXAMPLE 1.—A pile was driven with an ordinary hammer weighing 2,400 pounds. The sinking under the last five blows was 22 inches. The fall of the hammer during the last blows averaged 28 feet. What was the safe bearing power of the pile?

SOLUTION.—Here the value of s may be taken as the average of the total sinking during the last five blows, or $22 \div 5 = 4.4$ in. Then, $w = 2,400$ lb.; $h = 28$; and $s = 4.4$. Substituting these values in formula **1**,

$$R = \frac{2 \times 2,400 \times 28}{4.4 + 1} = 24,889 \text{ lb. Ans.}$$

EXAMPLE 2.—Piles are to be driven by a steam pile driver. The hammer weighs 5,500 pounds, and its fall is 40 inches. It is required that the piles shall be driven until each shall have a safe bearing value of 30,000 pounds. What must be the sinking under the final blow?

SOLUTION.—In this case R is given, the unknown quantity being s. From formula **2** the following is derived:

$$s = \frac{2\,w\,h}{R} - .1$$

In this case, $R = 30,000$; $w = 5,500$; and $h = 40$ in. $= 3.33$ ft. Therefore,

$$s = \frac{36,630}{30,000} - .1 = 1.2 - .1 = 1.1 \text{ in.}$$

Hence, the piles should be driven until their penetration for each blow is approximately 1 in. Ans.

EXAMPLES FOR PRACTICE

1. A pile is driven by a drop-hammer weighing 2,500 pounds, with a fall of 16 feet. The last blow of the hammer drives the pile 1 inch. What load will the pile carry safely, according to the Engineering News formula? Ans. 20 tons, nearly

2. It is desired that a pile shall be driven with a drop hammer, weighing 3,000 pounds and having a fall of 15 feet, until it shall be able to carry safely a load of 24 tons. What, by the Engineering News formula, must be the penetration at the last blow of the hammer?
Ans. .9 in., nearly

3. A pile, driven with a steam hammer weighing 5,000 pounds and having a fall of 42 inches, can safely carry a load of 32,000 pounds. What is the penetration at the last blow of the hammer?
Ans. 1 in., nearly

MISCELLANEOUS MATTERS RELATING TO PILE CONSTRUCTION

19. Pile Ends.—The lower end of a pile is usually, though not always, dressed down to a roughly conical point. Where wooden piles are driven in very hard material, or in gravel and stony ground, their points are liable to be battered and injured. To protect the point, it is often provided with an iron shoe. Several forms of shoes are shown in Fig. 5 (*a*), (*b*), and (*c*). It is also necessary or desirable to protect the heads of the piles, particularly when the drop hammer is employed and the fall is great. The most common device for this purpose is a heavy iron ring surrounding the head of the pile, as shown in Fig. 5 (*d*). The wood may be dressed down to receive the ring, but more commonly the ring is adjusted on the flat top of the pile and the hammer allowed to fall on it, forcing the ring into the wood. It is generally considered more desirable to use a heavy hammer with a low fall than a light hammer with a high fall. With high falls, the danger of splitting or breaking the pile or of unduly battering and brooming its head is greatly increased.

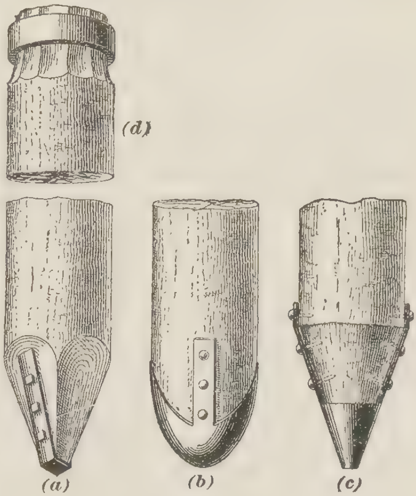


FIG. 5

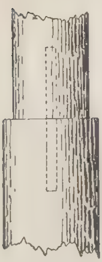


FIG. 6

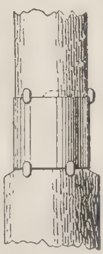


FIG. 7

20. Splicing Piles.—When a firm subsoil cannot be obtained except at a very great depth below the surface, so that it becomes impracticable, if not

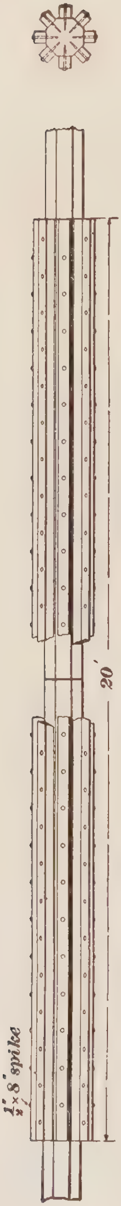


Fig. 8

impossible, to obtain timber piles of the requisite length, it may be necessary to splice the piles. In general, a **splice** is some form of joint connecting the ends of two piles. It may be a dowel or iron bar about 2 feet long, embedded about 1 foot in each pile, as illustrated in Fig. 6. A still more secure joint is a band of iron, at least 12 inches long and of as great a diameter as necessary, to which the ends of both piles are fitted. To prevent the possibility of the band being jarred either up or down, away from the joint, spikes are driven into the piles immediately above and below the band, as illustrated in Fig. 7.

When the ground is in a partly fluid state, it may have so little stiffness that the jointed pile is likely to buckle at the joint, unless the splice is made in the form of scarfs, which consist of six or eight timbers about 3 inches square in sectional area and 8 to 10 feet long, spiked to the piles, as shown in Fig. 8. Whatever form of splice is used, the first or lower pile is driven into the ground until its top is but a few feet above the surface; then the second pile is spliced on and driven in. Of course, the force required for driving in a soil so soft that so long piles are necessary is correspondingly small.

21. Cutting Off the Piles.—When a timber foundation is to be constructed on piles, it is necessary to cut off the tops of the piles to bring them to an even horizontal plane, so they may properly receive the caps or timbers that rest directly on them. When not submerged, the piles may be cut off by hand, but it is usually more expeditious and economical to do this by mechanical power. A circular saw at the end of a vertical shaft is commonly employed, the

frame that carries the shaft and saw being made adjustable in height and attached either to the leads of the pile driver or to some part of the platform carrying the pile driver. The power to operate the saw may be supplied by the hoisting engine of the pile driver. Where a concrete foundation is to be used in connection with the piles, it may not be necessary to cut off the piles, provided that they stand at approximately the same height and their heads are not so much broomed as to prevent a firm contact between them and the surrounding concrete; in all important foundations, however, it is generally advisable to cut off the piles.

22. Amount of Driving Necessary.—While it is important that piles should be driven to a firm seat and bearing in the soil, it is not advisable to overdrive them. It is not thought advantageous to continue hammering a pile after it has ceased to move about $\frac{1}{2}$ inch with each blow, and many engineers consider a pile sufficiently driven when it fails to move more than 1 inch with each blow. Piles driven until they will go no farther, or until they move but a small fraction of an inch per blow, are said to be "driven to refusal."

When piles are driven through clay to an underlying rock floor, driving should cease as soon as the point of the pile rests solidly on the rock; further hammering may batter the point of the pile and lessen its supporting power.

23. Spacing of Piles.—When a large number of piles is required, it is best not to attempt to drive them nearer together than about $2\frac{1}{2}$ feet from center to center, or about one pile to each 6 square feet of ground surface; if driven more closely, the great compression of the soil may tend to force upwards those already driven. The danger of this may be lessened if the center piles of the group are driven first and the work proceeds outwards from them.

24. Followers.—When piles are located in water and their heads must be driven below its surface, a short pile or timber, called a **follower,** is set up on top of the pile

when its head reaches the surface of the water, and the driving continued until the pile is sufficiently driven.

25. Withdrawing Piles.—It is sometimes necessary to pull or withdraw piles already driven, particularly where they have been used for the construction of coffer dams and similar work. Various devices have been used for the purpose. For short sheet piling, a powerful lever worked by hand is often sufficient; when this is ineffective, powerful block and tackle operated by a hoisting engine or other power is employed. The force required to withdraw a pile is about the same as the ultimate load the pile will support.

26. Timber Suitable for Piles.—Any timber strong enough to withstand driving will have sufficient strength for foundation purposes, and if the piles when in place are below the surface of the water or are deeply submerged in damp earth, they will prove sufficiently durable. But if their tops are to be exposed to the air or submerged in water but a part of the time, it is necessary to select species of timber that are not subject to rapid decay. It is often advisable to treat the timber chemically. For not very hard soils, spruce and hemlock make good pile material; for harder soils, the hard oaks, pines, and elm may be used.

SHEET PILING

27. While sheet piling is sometimes employed for carrying loads temporarily, its use primarily is to form an enclosure around a foundation area to shut out water, silt, etc. For this purpose, sawed timbers or plank are nearly always used. When strength to support great pressure is not required, ordinary plank from 3 to 4 inches thick will be sufficient. When greater strength is required, square timbers may be used, but more frequently two or more thicknesses of plank are employed. The purpose for which sheet piling is used makes it important that the joints between adjoining piles should be as close and as nearly water-tight as possible. Several devices for securing this condition are

in use, a number of them being shown in Fig. 9. In this figure, (*a*) represents the plain sheet piling, which depends for close joints on the care and accuracy with which it is driven. In (*b*) are shown two contiguous rows of piling, one row breaking joint with the other; three rows of plank are sometimes used, the outside rows being of thinner plank than the central or main row. In (*c*), (*d*), (*e*), (*f*), and (*g*) are shown various forms of "tongued-and-grooved" sheet piling; the object of this construction is not only to make a

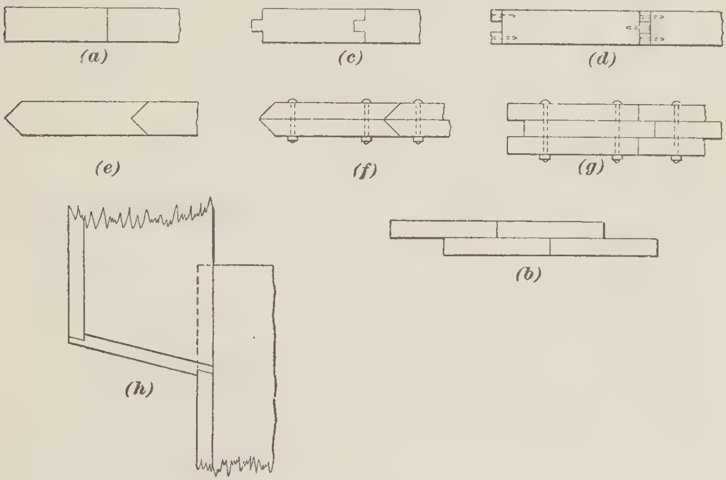


Fig. 9

tight joint, but also to assist in guiding the piles truly in the same plane. The form (*d*) is very frequently used, the tongue and groove being formed by spiking narrow strips on the edge of the pile. The form (*g*) is a patented form known as the "Wakefield" pile; its construction is evident from the figure without further description. The points of sheet piles are usually sharpened to facilitate driving, and one side is often sloped, as shown in (*h*), to cause the piles to come into close contact with one another.

CONSTRUCTION OF PILE FOUNDATIONS

28. When land foundations are to be carried to great depths to reach solid rock or other strong subfoundation, various expedients are in use to avoid the excavation of deep continuous trenches, which are expensive and often troublesome. The simplest expedient is a pile foundation. The depth to which the piles are driven depends on the character of the soil and the weight to be carried by the piles. This may best be determined by the experimental driving of a few piles, though test pits or borings will generally indicate pretty closely the probable length required. The true criterion is the resistance of the piles to movement with the last few blows of the hammer (see Art. **18**). A pile foundation is really a subfoundation, which must be connected to the structure by a foundation proper. Two principal methods are used for this purpose; namely, by *timber grillage*, and by *concrete filling*.

29. Timber Grillage.—A **grillage** consists of two or more courses of timber laid at right angles to each other, each course consisting of parallel timbers laid at intervals usually not much greater than the width of the timber. Grillage is often used as a foundation on a soft-soil subfoundation or on piles. When used with piles, the latter, properly spaced, are driven in rows, as many rows being placed as are required to cover properly the foundation area.

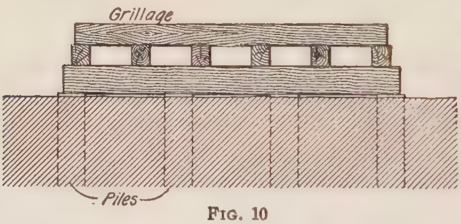


FIG. 10

The tops of the piles are cut off to a horizontal plane as near the earth as practicable, and a line of timbers, called **caps**, are placed on the tops of the piles along each row, and are secured to them by drift bolts.

It is necessary to exercise care to have the piles truly in line in the rows, in order that the caps may have a full bearing on the tops of the piles. On the caps, and at right

angles to them, another set of timbers, spaced from 1 to 3 feet apart, is placed and drift-bolted to the caps. Several such courses of timbers may be used if desirable, each being placed at right angles to the previous course. This construction is illustrated in Fig. 10.

30. Concrete Filling.—In concrete filling, it is not so essential that the piles be driven accurately in rows, nor that their tops be cut off, if they are all nearly at the same level as driven; although, as the tops are likely to be more or less battered or broomed by driving, a better connection between them and the concrete will be secured by cutting them off, or at least by removing the broomed part with axes or adzes. This being done, the loose or soft portions of the earth are removed so as to give the concrete a good bearing on it, and the concrete is deposited between and around the piles and carried to a height of at least 1 foot above their tops, where it is formed into a level surface for the reception of the masonry.

ORDINARY MASONRY FOUNDATIONS

31. General Principles and Methods.—The construction of **ordinary masonry foundations,** consisting of superimposed courses of masonry either with or without footings, does not present any special difficulty. The required area of subfoundation, and the dimensions of the footings, if any, are computed by the principles explained in *Foundations*, Part 1.

32. Excavation: Sheeting.—In many cases, foundations are carried to a depth just sufficient to secure firm material, or to be below the disturbing action of frost. Frequently, however, the subfoundation material near the surface is unsuitable, or the unusually heavy loads to be carried necessitate going down to a strong stratum of material.

Very deep excavations, even in soils that are tolerably dry and firm, are liable to cave in, unless made with very flat slopes, or unless the banks are sustained by artificial supports. The means of support most commonly employed is called **sheeting.** It consists of a lining of plank held in

place by longitudinal stringers, and cross-struts that brace one side of the excavation against the other. The method of excavating by sheeting is illustrated in Fig. 11, where *a*, *a* represent the planking, *b*, *b* the longitudinal stringers, and *c*, *c* the struts. Between the ends of the struts and the stringers, wooden wedges are driven to hold the planking tightly against the earth wall, or one end of the struts is fitted with a jack-screw by which any desired degree of pressure can be applied.

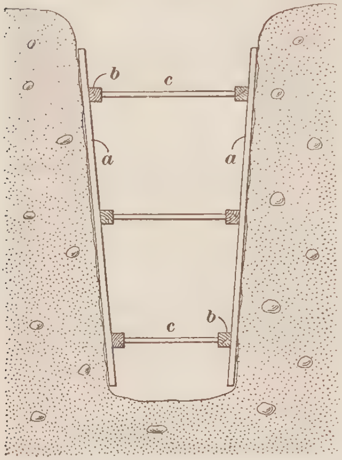


FIG. 11

33. Base Course.—The most satisfactory base course for a land foundation is a bed of concrete. It is comparatively economical; the mortar when placed accommodates itself to any irregularities of the subfoundation; and, when hardened into a monolithic mass, the bed has sufficient strength to distribute the loads properly and to bridge over any soft places in the subfoundation. If the excavated trench is cut to accurate dimensions, no forms or molds are necessary to hold the mortar in place.

34. Removal of Water.—In all land foundations of considerable depth, and in shallow ones in swampy soils, more or less water is likely to be encountered, and must be drained or otherwise disposed of. Frequently, the quantity is so small that it may be removed by ordinary buckets, which were probably the earliest devices for the purpose. For this reason, doubtless, the removal of water from such foundations is called "bailing and pumping." The cost of removing water often constitutes a large percentage of the whole cost of the foundation. Where the quantity of water is large and it must be raised to a considerable height, power pumps are employed.

TUBULAR FOUNDATIONS

35. Tubular foundations, called also **curb foundations**, are frequently used. They consist of a tube or curb of any desired size, either cylindrical or square, built of steel, cast iron, or wood. The method of sinking the tube is illustrated in Fig. 12, where *A* represents a cylinder built of sheet steel. A well, somewhat larger in diameter than the tube, is first excavated to as great a depth as possible without support for the walls. The cylinder is then placed in the well, and the excavation continued inside it, the earth being removed from under its edge, as shown, so that the cylinder sinks either by its own weight or by loads placed on its top, following the excavation downwards to the required depth; additional lengths are riveted to the top as necessary. The friction of the outer surface of the cylinder against the earth walls increases, of course, with the depth, and may limit the depth attainable. This friction varies from 100 to 150 pounds per square foot of surface, depending on the depth and the coefficient of friction between the cylinder and the material through which it is sunk. When the friction becomes so great that the cylinder can be forced downwards no farther, a second cylinder, smaller in diameter than the first, may be started in the bottom of the excavation and sunk in the same way as the first. Any number of additional cylinders may be thus used, and the excavation carried to any desired depth, provided that the first cylinder is made of sufficient diameter.

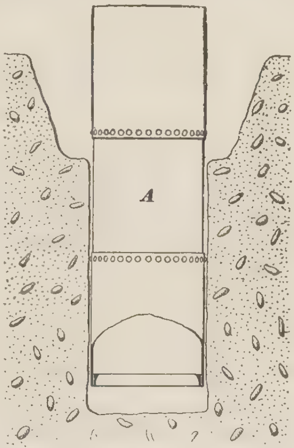


Fig. 12

36. If too much water is encountered to be conveniently handled by pumps, some form of dredge bucket may be used

to carry on the excavation. After the tube has reached the bed rock or other satisfactory bearing material, it is filled with concrete or brick masonry, and a suitable bearing cap is placed on its top. A sufficient number of these tubes being in place to support the proposed structure, steel beams are placed over them, or they are connected by brick or concrete arches. Where water is encountered, these tubes may be fitted with air locks and sunk by the pneumatic process, as will be described further on. Wooden cylinders lined with brick were the earliest representatives of this class of work, and have been sunk to very considerable depths.

FOUNDATIONS FOR BUILDINGS

37. General Considerations. The construction of adequate foundations for the great modern buildings of cities presents difficult and peculiar problems, which have developed a special class of foundation work. Large cities are built on geological formations differing widely in their physical characteristics, and, as might be expected, there has been developed in each a general method of foundation construction best adapted to its peculiar conditions. The characteristic foundation in Chicago, for instance, is different from that in New York. Certain general conditions and principles are, however, common to all, and a brief consideration of these is all that can be given here.

38. The restricted area and the great cost of city real estate in the business districts make it necessary that every available square foot of each building lot be enclosed and utilized for business purposes. The walls of the building therefore extend fully to the boundary lines, and their foundations can be extended only inwards. Even in that direction, basement and cellar space is so valuable that it may be given up for foundation purposes to such extent only as is absolutely unavoidable. This scarcity of space and the enormous cost of land have led to the construction of exceedingly high buildings, and this in turn has resulted in concentrated

foundation loads heretofore unprecedented. At the same time, the walls of buildings are frail compared with most other engineering structures, and any uneven settlement of the foundation is liable to result in cracks and disfigurements.

39. Subfoundation.—To support the enormous weights that must be carried, the first and most obvious expedient has been to carry the foundation down to solid rock or to a stratum of hard pan or compacted gravel having the necessary bearing resistance. Where this is not practicable, and a compressible subfoundation must be used, the aim has been to spread the foundation over an ample area, and to so distribute it with reference to the varying weights of different parts that the pressure per unit area will be as nearly as possible the same. To avoid eccentric loading of the foundations, various devices have been used, some of which will be described presently.

In cities where compressible subfoundations are used, the foundations are placed at a sufficient depth to be below the basement, and are spread over a large area. Great care is exercised to make the intensity of pressure the same under every part of the foundation.

CANTILEVER FOUNDATIONS

40. To obviate eccentric loading, and to transfer loads from points where support is not practicable to other points, steel beams, especially cantilevers, are extensively employed. An example of their use is illustrated in Fig. 13, which represents the wall of a new steel-frame building placed directly against the wall *b* of an old building. If the column *f* rested directly on the foundation *a*, it would produce eccentric loading. To provide against this eccentric loading, the cantilever beam *d* is introduced, which transfers the center of pressure from *c* to *e*, directly over the center of the foundation. This example may be regarded as typical of most cases of the employment of cantilever beams in foundation work, and its conditions may be briefly investigated.

Let W = weight on column f;
R_1 = reaction of foundation at e;
R_2 = reaction of column h, to which the cantilever d is riveted.

The meaning of the letters l and m is shown in the figure. Since the cantilever is in equilibrium under the action of the

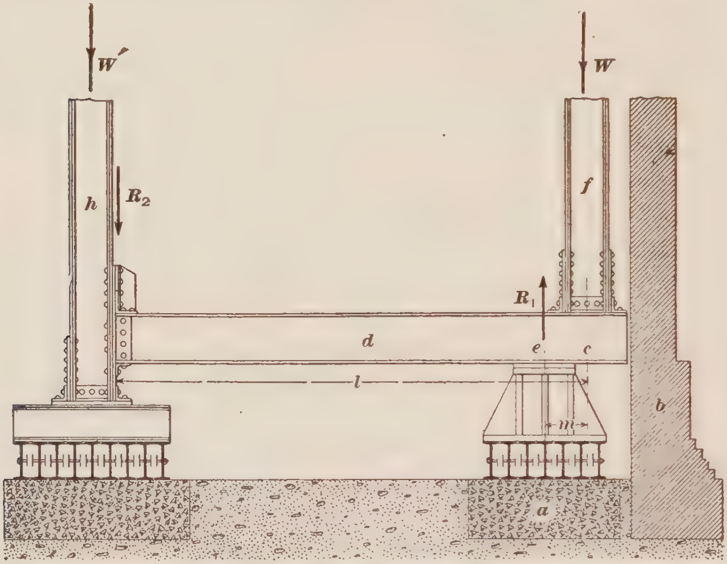


FIG. 13

forces W, R_1, and R_2, we have, taking moments about the left-hand end of the cantilever,

$$Wl = R_1(l - m);$$

whence $$R_1 = \frac{Wl}{l - m} \qquad (1)$$

Taking moments about e,

$$Wm = R_2(l - m);$$

whence $$R_2 = \frac{Wm}{l - m} \qquad (2)$$

The cantilever exerts on the column h an upward force equal to R_2, and diminishes the pressure of the column h on its foundation. It may occur that R_2 is greater than the

weight W' on column h, in which case the resultant pressures on the foundation under that column will be negative, and the column must be anchored to its foundation to prevent its being lifted from its bearing.

CONTINUOUS FOUNDATION UNDER TWO COLUMNS

41. While it is desirable, with compressible subfoundations, that each pier or column should have its independent foundation, it may happen that two or more such piers or columns located near each other must be placed on the same foundation. In such a case, the important requisite is that the resultant of the downward pressure of the two columns shall coincide with the center of gravity of the subfoundation. If the weights on the columns are different, it is important also that the foundation shall be so designed as to secure an equitable distribution of the stresses throughout it. These conditions cannot always be secured, but may be approximated.

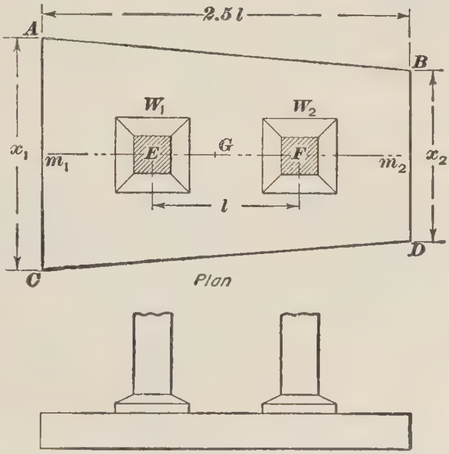


FIG. 14

With two columns carrying different weights, a trapezoidal form of foundation is naturally suggested. Such a foundation may be designed by the following method:

In Fig. 14, let $A B D C$ represent a trapezoidal foundation carrying the two columns E and F, the distance between their centers being l. Let the weights carried by the columns E and F be W_1 and W_2, respectively, W_1 being the larger. The total weight on the foundation being $W_1 + W_2$, if the subfoundation may be safely loaded with s tons per square foot,

the required area A will be $\frac{W_1 + W_2}{s}$. The length of the foundation should preferably be not less than $2\,l$ nor more than $3\,l$. The widths of the ends of the trapezoid may be made proportional to the loads W_1 and W_2. If the width at the wide end is denoted by x_1 and that at the other end by x_2, then,

$$\frac{x_1}{x_2} = \frac{W_1}{W_2};$$

whence
$$x_2 = \frac{W_2 x_1}{W_1}$$

If the whole length (altitude of the trapezoid) of the subfoundation is made $2.5\,l$, then,

$$A = \frac{2.5\,l}{2}(x_1 + x_2) = 1.25\,l\left(x_1 + \frac{W_2 x_1}{W_1}\right)$$
$$= 1.25\,l\,x_1\left(\frac{W_1 + W_2}{W_1}\right);$$

whence
$$x_1 = \frac{A\,W_1}{1.25\,l(W_1 + W_2)}$$

Or, since $A = \frac{W_1 + W_2}{s}$,

$$x_1 = \frac{W_1}{1.25\,l\,s} = \frac{4\,W_1}{5\,l\,s} \qquad (1)$$

Also,
$$x_2 = \frac{W_2 x_1}{W_1} = \frac{4\,W_2}{5\,l\,s} \qquad (2)$$

The center of gravity G is determined by one of the methods explained in *Analytic Statics*, Part 2.

Let the distances $E\,G$ and $F\,G$ be denoted by y_1 and y_2, respectively. In order that the resultant of W_1 and W_2, which is $W_1 + W_2$, may pass through G, we must have, taking moments about E,

$$(W_1 + W_2)\,y_1 = W_2\,l;$$

whence
$$y_1 = \frac{W_2\,l}{W_1 + W_2} \qquad (3)$$

Similarly,

$$y_2 = l - y_1 = \frac{W_1\,l}{W_1 + W_2} \qquad (4)$$

The distances $m_1 E$ and $m_2 F$ can now be easily computed, located, and staked out.

EXAMPLE.—Suppose that in Fig. 14 the distance between the centers of the columns E and F is 8 feet, and the loads are 200 and 150 tons, respectively. If the length of the trapezoid is 20 feet and the capacity of the subfoundation 1.5 tons per square foot, what is the distance $E m_1$?

SOLUTION.—To apply formulas **1** and **2**, we have $W_1 = 200$, $W_2 = 150$, $l = 8$, and $s = 1.5$. Substituting in the formulas,

$$x_1 = \frac{4 \times 200}{5 \times 8 \times 1.5} = 13.33 \text{ ft.}$$

$$x_2 = \frac{4 \times 150}{5 \times 8 \times 1.5} = 10 \text{ ft.}$$

It is found that the center of gravity of the trapezoid is 9.5 ft. from $A C$.

The distance of E from the center of gravity of the trapezoid is found by formula **3**:

$$y_1 = \frac{150 \times 8}{200 + 150} = 3.43 \text{ ft.}$$

Therefore, $E m_1 = 9.5 - 3.43 = 6.07$ ft. Ans.

FOUNDATIONS UNDER WATER

42. Where the foundation of a structure must be placed in water, one of two general methods is followed: either the space to be occupied is freed of water, after which the work proceeds as on land, or some device for sinking the foundation through the water is employed.

COFFER-DAM PROCESS

43. Coffer Dams.—A **coffer dam** is a water-tight structure enclosing a space from which the water may be pumped out, leaving a comparatively dry area on which a foundation may be constructed as on dry land. A great variety of coffer dams have been devised and used. The simplest and most primitive, suitable for use in very shallow water, is a bank of earth, clay, or sand, rising above the surface of the water, surrounding the area to be built on.

Next in order is a simple timber structure surrounded by a bank of earth. Fig. 15 shows a cross-section of a coffer dam of the latter form. A wall of sheet piling is driven around the space to be unwatered, the tops of the sheet piles

are spiked to a timber, called a **wale** or **waling piece,** and a bank of earth is then deposited around the outside of the structure.

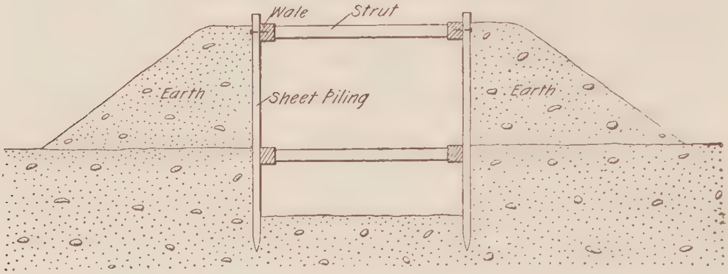


FIG. 15

44. A form of coffer dam very frequently used is illustrated in Fig. 16. It consists of an inner and an outer line of sheet piling with waling pieces at the top, tied together with iron bolts. Frequently, each wall is composed of two or more lines of sheet piling, as shown on the left-hand side, the individual piles breaking joint for the purpose of increasing the strength of the structure, and of making the walls

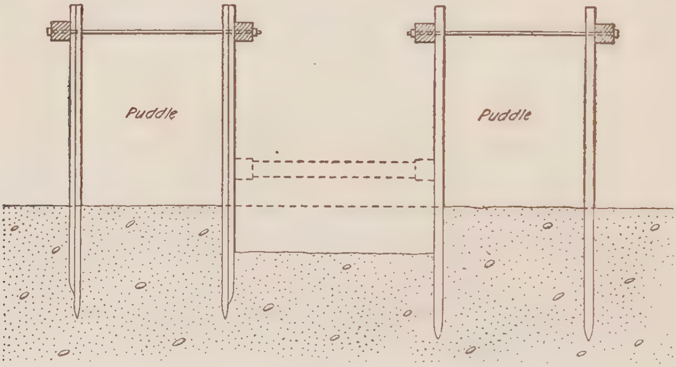


FIG. 16

themselves as nearly water-tight as possible. The space between the two walls is then filled with earth or other suitable material to exclude the passage of water. This material is called **puddle.** The water may then be pumped from

the interior, and, if necessary, additional wales and cross-struts, as shown by dotted lines, may be added as the water is lowered, to resist the pressure inwards.

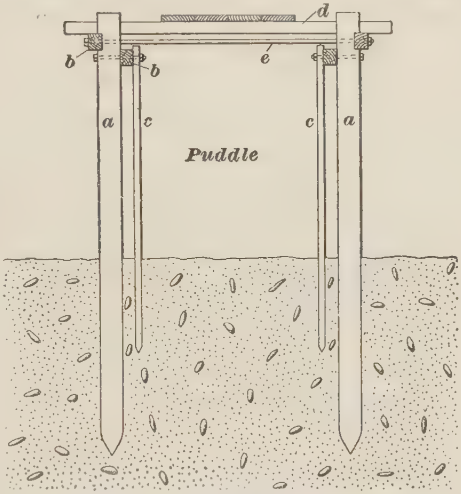


FIG. 17

45. For use in still deeper water, particularly where there is a strong current, the construction illustrated in Fig. 17 is employed. In this figure, a cross-section of one wall is shown. Two rows a, a of ordinary piles, with a suitable space between them, are first driven around the area to be enclosed. Wales b, b are then bolted to the sides of these piles, the outer lines being placed somewhat higher than the inner ones, to support cross-timbers d used for carrying a floor or runway. Two rows c, c of sheet piling are then driven along the inner wales, to which the tops of the sheet piling are spiked. Long bolts e, extending through the outer wales, prevent the spreading of the structure by the pressure of the puddle. The space between the sheet piling is then filled with puddle.

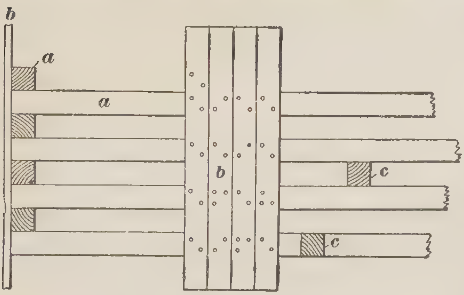


FIG. 18

46. Another typical form of coffer dam is constructed of cribwork, as illustrated in Fig. 18. It consists of a wall of timber cribwork built of square timbers a, a drift-bolted

together at their intersections, and a sheeting of plank *b*, *b* spiked to the outer side of the wall. The illustration shows the details with sufficient clearness, and no further description is necessary.

Coffer dams of this kind may be built on land, launched, floated into position, and sunk to the bottom by appropriate means. If the sheeting is fairly well seasoned, with dressed edges, and the plank is put on with as close joints as possible, the swelling of the wood when the structure is placed in the water will make the joints practically water-tight. Sometimes, the timbers *a*, *a* are cut half away at the corners so that the parallel sticks are in close contact, and the joints are then calked to make them water-tight. Cross-timbers *c*, *c* may be put in occasionally to brace the sides of the coffer dam against each other.

47. Design of Coffer Dams.—The two most important requisites in a coffer dam are that it shall have sufficient strength to resist the pressure of the water on the outside when that on the inside is pumped out, and that it shall be water-tight. Since the intensity of the pressure increases with the depth, not only must the lower parts of the structure be made stronger, but greater care must be taken to prevent leakage. The total pressure P, in pounds per lineal foot, on the side of a coffer dam is found by the following formula (see *Hydrostatics*):

$$P = \frac{62.5\,h^2}{2}$$

in which h is the depth, in feet, of the submerged part of the coffer dam.

The intensity of pressure at any depth h_1 below the surface is $62.5\,h_1$ pounds per square foot, and the center of pressure is at a depth of $\frac{2}{3}\,h_1$ from the surface.

The stresses on coffer dams and their several parts are determined by the application of the mechanics of beams. Where main piles are driven, as in Fig. 17, they act as cantilever beams; where the surface material into which they are driven is soft, their capacity is decreased, and their strength should be figured with a large factor of safety.

The sides of the coffer dam may be braced against each other as shown in the preceding figures, but too many struts of this kind obstruct the laying of the masonry, and it is accordingly desirable to use as few of them as possible. As the masonry reaches them, they may be cut away and short struts resting against the masonry substituted.

48. The necessary thickness of the wall of puddle will usually determine the width of the double walls of the coffer dam. No definite rule can be given for the thickness of the puddle wall, as it depends so largely on the character and efficiency of the puddle material available. It should never be less than 4 feet, even in shallow water. An empirical rule given in Trautwine's "Pocketbook" is to make the thickness three-fourths of the height, but never less than 4 feet. This rule seems to give good results with puddle of average quality and for moderate depths. For use in water 12 feet deep, the rule would require a width of 9 feet of puddle, which is probably more than most engineers would think it necessary to use.

49. Quality of Puddle.—The effectiveness of a coffer dam in excluding water depends not only on the thickness of the wall of puddle, but also on the quality of the puddle itself. Stiff, tenacious clay, capable of being rammed into a dense mass, would seem at first thought to be the most desirable material. It is often used with success, but it has the defect that, if the water once penetrates it in a stream, however small, the current will continually enlarge the opening, the stiffness of the clay preventing it from falling into and closing the channel. Experience has shown that a mixture of gravel, sand, and clay makes the most effective puddle material, the clay being barely sufficient in quantity to fill the voids in the gravel and sand. If from any cause a rill of water should start through a mass of this material, the gravel and sand will cave into the channel and close it up effectively.

50. Water-Tightness of the Structure.—When the coffer dam is complete, pumps are put in operation to

remove the water from its interior. This affords the first test of the water-tightness of the structure, and, if it is not satisfactory, the leaks must be looked for, and such expedients as the conditions suggest must be applied to stop them. It will seldom be found that a coffer dam is perfectly tight, but, if the leakage is not too large to be controlled by the pumps, the latter may be kept in operation and the water thus kept down so as to permit the construction of the foundation.

51. Several forms of interlocking steel sheet piling are now available, and are particularly valuable for coffer-dam work. This piling has a much greater strength than wood piling; the interlocking feature insures a practically water-tight joint between adjoining members; and, although its first cost is much greater than that of wood, it may often prove more economical, particularly in deep water.

52. Deep coffer dams, exposed to great pressure of water, require special care to make them water-tight. Even if the walls do not leak, the water may be forced under them or through the soil beneath them, unless the sheet piling extends to a considerable depth and the separate piles are driven in close contact with one another for their whole length. To secure the latter requisite, some form of tongued-and-grooved sheet piling is generally used, as described in Art. **27**, and it is driven, if possible, to the rock or other hard and impervious material which is to form the subfoundation. Even then the water may rise through subterranean passages or through fissures in the rock in such quantities as to be very troublesome. Such cases must be dealt with as the conditions and possibilities suggest. Sometimes, a leak may be totally or partly stopped by depositing a bank of puddle around it of sufficient width to blanket and close off any water passages in the near vicinity of the work.

53. Dredging.—If the stratum it is proposed to use for subfoundation is overlaid with soft and pervious material, the whole area to be occupied by the coffer dam is often dredged away before the coffer dam is built. Where crib

coffer dams like that shown in Fig. 18 are used, they may often, after they are in place, be sunk to a considerable depth by dredging out the interior, allowing the coffer dam to sink by its own or by added weight. This method is similar to that used for sinking crib foundations, which will be described further on.

PNEUMATIC PROCESSES

THE USUAL PROCESS

54. General Description.—The carrying of foundations to great depths under water or in water-bearing soils involved very serious difficulties until the development of the **pneumatic process.** By the aid of this process, foundations are now carried to depths as great as 100 feet below the surface with safety and the certainty of success. This process utilizes air pressure to facilitate the sinking of a foundation structure, called a **caisson,** through the soil, to bed rock or other hard subfoundation; the proposed structure is built up on this caisson. In the earlier pneumatic foundation work, a partial vacuum was created in the closed caisson, and the difference of pressure between the outer air and that in the caisson was utilized to force the latter into the ground. This vacuum process is now seldom used, and need not be further considered.

55. At present, compressed air is used principally to counteract the pressure of the water, and prevent the latter from flowing into and filling the caisson. The principles involved and the practical working of the process can best be explained by reference to the illustration in Fig. 19, which shows a vertical cross-section through the caisson. The caisson itself may be likened to a box without a lid, placed upside down. Its sides are formed of several thicknesses of heavy timbers tied together with screw and drift bolts, the bottom part being of triangular shape, forming a sharp edge, called the **cutting edge,** which is usually shod with iron. The cover, called the **deck** of the caisson, is formed of

several, often a dozen or more, courses of timbers crossing each other at angles usually of 90°, and the whole is thoroughly tied together with bolts. In large caissons, one or more trusses or partitions crossing from side to side give the structure additional strength and stiffness. The interior open space is called the **working chamber.** The walls and roof of this chamber are made as nearly water-tight and air-tight as possible by lining them with courses of plank, the joints between which are calked with oakum

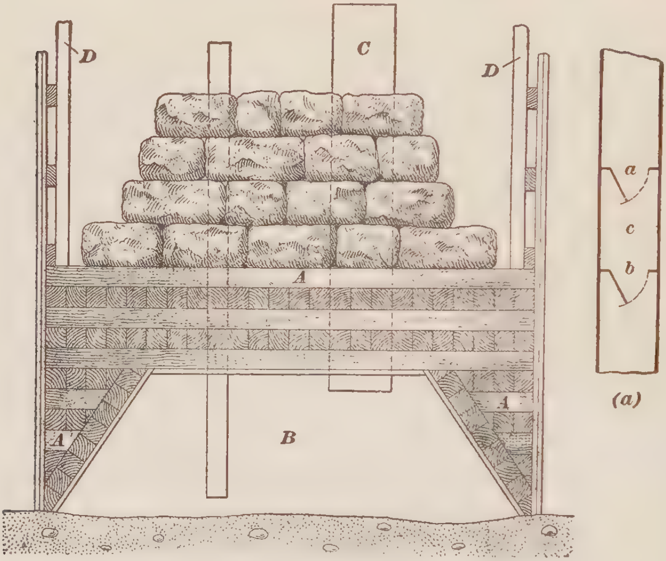


Fig. 19

and pitch. On the deck is usually, though not always, constructed a water-tight enclosure $D\,D$, called the **coffer dam,** the object of which is to permit the laying of the masonry below the surface of the water, if found desirable. This coffer dam may be but a few feet in height to protect the laying of the first few courses of masonry, or it may be carried to the full height of the surface of the water when the caisson has reached its permanent position. Through the deck, communicating with the working

chamber, are inserted a number of large tubes or shafts built of steel plates, which are carried up as the sinking of the caisson and the building of the masonry progress. The most important of these shafts is called the **air-shaft,** which is used for the entrance and exit of the men to and from the working chamber, and sometimes for the bringing out of excavated material. Two or more such air-shafts may be provided in large caissons. Situated in the air-shaft is the **air-lock,** the use of which will be explained presently. Other tubes and pipes are for the forcing out of water and semifluid excavated material, or for conveying concrete and other materials into the working chamber.

Referring to Fig. 19, *A*, *A*, *A* represents the caisson structure; *B* is the working chamber, and *C* is the air-shaft. These air-shafts are constructed with differing details, but the principle involved is the same in all.

The principle of the air lock will be understood from Fig. 19 (*a*). The lock consists of a chamber *c* separated from the upper and lower portions of the air-shaft by doors *a* and *b*, which are fitted to close air-tight. The air pressure in the working chamber and in the lower part of the air-shaft may be from 10 to 40 pounds per square inch higher than that in the upper part of the air-shaft, which is in open communication with the atmosphere. A man wishing to enter the working chamber passes down the air-shaft through the upper door of the air lock, the lower door being closed. When he is inside the air lock, the upper door is closed, the compressed air from below is slowly admitted into the lock through valves for the purpose, and when the pressure in the lock has become equal to that in the working chamber, the lower door is opened and the man continues his descent. If he wishes to return, the operation is reversed: the upper door being closed, the lower one is opened; he enters the lock and the lower door is closed; the lock is then connected through valves with the atmosphere; the compressed air in the lock escapes until the pressure is the same as that outside, when the upper door is opened and the man continues his ascent.

56. Sinking the Caisson.—Pneumatic caissons may be used either on land or in water. If for use in water, the structure may be built on land, launched when completed, floated to the site, and moored into the exact position where it is to be sunk. Mooring or guide piles are usually driven to control its sinking until it rests on the bottom. The laying of masonry on the deck is begun, and, as it progresses, the additional weight gradually sinks the whole structure until it rests on or sinks into the clay, sand, or mud of the bottom. The operation of further sinking the caisson is conducted as follows: When the walls of the caisson have penetrated the bottom material so as to seal the working chamber, air is gradually forced into it through a pipe provided for the purpose, driving out the water either under the walls of the caisson or through pipes passing upwards through the deck. The air pressure being maintained, the water is prevented from reentering the chamber, and the laborers descend through the air-shaft and begin the work of excavation. During this time, the building of the masonry has continued, the increasing weight being partly supported by the pressure of the air below and partly by the material under the edge of the walls of the caisson. As the excavation progresses, the material being removed from under the edge of the caisson sides, and the weight of the masonry on top of it increasing, the whole structure gradually sinks until the rock or other hard material is reached, the air pressure employed increasing with the depth so as to balance the water pressure from without.

57. The material to be excavated within the working chamber may be handled with picks and shovels, or by any of the other usual methods. If large boulders are encountered or if the caisson strikes solid rock on one side before the other, blasting may be employed.

Some care and skill are required to guide the caisson and keep it level. Various devices have been successfully used to move the structure horizontally even after it has been deeply buried. If from any cause it gets out of level, the

material is excavated more freely from under the cutting edge of the high side, or the load on that side is increased to force it down more rapidly.

58. Removing Excavated Material.—Formerly, the excavated material was carried out through the air-shaft, but in the more recent work only rock, boulders, and such material are removed in this way. Sand and ordinary gravel are forced out through a pipe, called a **sand lift**, passing up through the deck of the caisson and through the masonry, its lower end reaching to the floor of the working chamber. The excavated material, mixed with water to make it semifluid, is deposited around the mouth of this pipe, and the compressed air forces it in a continuous stream upwards through the pipe, which carries and discharges it wherever it is to be disposed of.

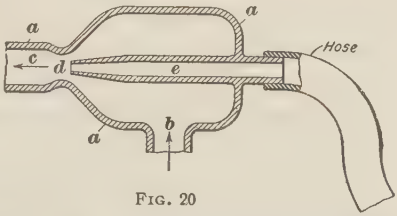


FIG. 20

59. Another very effective device for removing mud and sand is the **mud-pump**, which works on the principle of the common ejector. The current of water to operate it is supplied by powerful pumps outside the caisson. One form of mud-pump is illustrated in Fig. 20. It consists of an outer shell *a a a*, having an inlet *b* and an outlet *c*, the latter having a conical throat *d*. Entering the shell opposite this outlet is a pipe *e*, with its open end or nozzle terminating in the throat of the shell. The other end of this pipe is connected to a flexible pipe or hose, which may be moved about as required. When a stream of water under considerable pressure is forced through the pipe *e*, it escapes with high velocity through the pipe *c*, creating a partial vacuum in the shell *a a a*, by which the material is drawn in through *b*, carried forwards into the forced current of water, and discharged as in the sand lift previously described. The material fed to the mud-pump must be broken up or pulverized and mixed with sufficient water to make a semifluid mud.

60. Finishing the Foundation.—When the caisson has been carried down to the rock or other hard material on which it is designed to rest, and its cutting edge is firmly seated thereon, the working chamber is filled with concrete, or sometimes with sand, the shafts and tubes are filled and closed, and the foundation is complete.

61. General Remarks.—The caisson method of sinking foundations is now considered more satisfactory, efficient, and economical than any other for large foundations that must be carried to depths of 40 feet or more. Its application is limited, however, to depths of about 100 feet below the surface of the water, as the human system cannot endure air pressures due to greater heads. Formerly, the sickness and mortality among laborers made work in the working chamber a dangerous occupation. The **caisson disease**, as the peculiar disease caused by this work is called, was not then well understood. With proper hygienic care, it is now of infrequent occurrence, and seldom fatal.

When the working chamber reaches such a depth that the air pressure is more than two atmospheres, laborers can work only a limited length of time; and when the depth is over 90 feet, they should not be exposed to the pressure for more than 2 hours at a time, nor for more than two such periods, or "shifts," in each 24 hours.

62. The pneumatic-caisson process may be used also on land, when underground water makes any other process difficult or ineffective. The procedure is essentially the same as previously described.

The plant required for the pneumatic-caisson process is rather extensive. It embraces a complete outfit of boilers, powerful air compressors, and force pumps, together with the usual outfit for laying masonry and disposing of excavated material. In marine work, this plant is usually placed on barges moored around the work.

63. It is impossible to compute, in advance, with any degree of accuracy, the quantity or weight of masonry that will be necessary to force the caisson downwards,

as the conditions are likely to vary in different cases. The forces that resist the downward movement are the air pressure in the working chamber and the friction of the sides of the caisson on the walls of earth through which it passes. The air pressure required to keep the chamber free from water is somewhat less than that required to balance the column of water above the cutting edge, because of the friction of the water in passing through the material directly under the cutting edge. The friction of the caisson on the earth surrounding it appears to vary within very wide limits, but it is usually much less than the ordinary friction between wood and clay. The presence of water between the two surfaces, and the escape of more or less air, which naturally follows up along the side of the caisson, tend to reduce greatly the usual coefficient of friction. The records show that this frictional resistance may vary from 300 to 600 pounds per square foot of the exposed surface of the caisson.

The rate of sinking also varies with varying conditions, but mainly with the speed with which the material may be excavated in the working chamber and disposed of. If the caisson can be lowered $1\frac{1}{2}$ feet per day, working 24 hours, this may be regarded as good progress.

OTHER PNEUMATIC PROCESSES

64. Pneumatic Piles.—When the expression *pneumatic foundations* is used, the system just described is usually referred to, unless otherwise stated. However, other pneumatic methods, some of them older than the one prevailing now, are sometimes used. The name **pneumatic piles** is commonly applied to hollow cylinders sunk by the pneumatic process. The cylinders may be of any desired diameter, and may be made of riveted sheet metal, or of cast-iron rings or segments with inside flanges to bolt them together. The cylinders are similar in all respects to the tubes described in Art. 35, except that they are provided with air locks, which are placed near the bottom of the tube,

as this position offers the advantage that additional sections can be added to the top of the cylinder as it sinks, without disturbing the air lock. The bottom of the cylinder is usually provided with a stiffened cutting edge, and is flared out slightly to make a space in the clay somewhat larger than the body of the cylinder, in order to reduce the friction against its sides. Other details, and the method of sinking, do not differ materially from those of the pneumatic caisson. The filling of the cylinder with masonry is not begun until the cylinder has been sunk to its final position; then sufficient concrete is placed in the bottom to seal it against water, the air lock is removed, and the remainder filled in the open air.

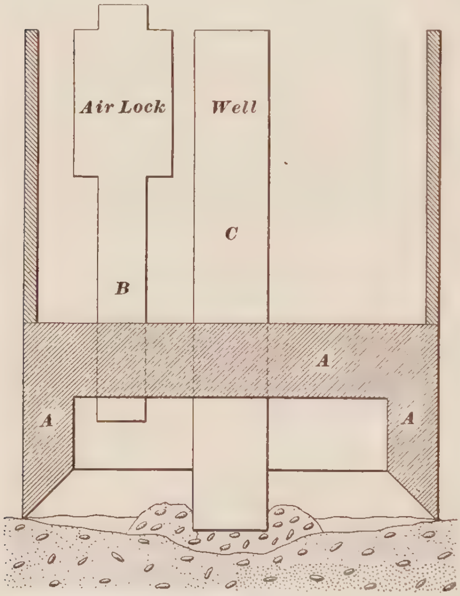


FIG. 21

65. It will be observed that these cylinders have not, as has the pneumatic caisson, the weight of the superimposed masonry to force them down, and difficulty is sometimes experienced in causing them to sink as rapidly as the material inside is excavated. Heavy weights may be placed on the top of the cylinder to assist in forcing it down, or the vacuum process may be employed to facilitate the sinking. If the friction is great and the cylinder fails to sink, a door opening upwards in the air lock may be closed and the air pumped out to form a partial vacuum in the cylinder, when the greater air pressure on the top of the air lock will force the cylinder down.

66. Water Column.—Another modification of the pneumatic-caisson process, formerly much used, relates to the method of removing the excavated material. Its principle will be understood by referring to Fig. 21, in which AAA is the caisson and B the air-shaft, with its air lock for the ingress and egress of the workmen. A well, or **water column,** C extends through the caisson to a depth a little below the plane of the bottom of the caisson. This well is filled with water to a sufficient height to balance the air pressure in the working chamber. The excavated material is piled around the well, into which it flows; it is then removed by some form of dredge like the common clam-shell bucket. The superior capacity and greater economy of the sand lift and the mud-pump, as now developed, for the removal of excavated material is such that the water-column method which formerly was much used, is not extensively used at present.

RIPRAP FOUNDATIONS

67. Simple Riprap Foundations.—Among the methods used to place foundations in water without freeing the site of water, riprap foundations are the most simple and primitive. **Riprap** is a name applied to a mass of broken rock deposited at random, or with little or no attempt at regular arrangement. Its use is commonly restricted to the protection of river banks or insecure foundations from erosion or undermining, but many foundations for bridge piers have been constructed with this material. Masses of rock as large as can be conveniently handled are thrown overboard from barges until a pile is accumulated reaching to, or nearly to, the surface of the water and having a top area considerably larger than the base of the pier. The surface is rudely leveled off with smaller fragments, and the masonry begun without further preparation. Settlement, sometimes irregular, will occur as the masonry progresses; this may be partially corrected by varying the thickness of a course of stone. Later, additional riprap may be deposited, enlarging the mass and building it up a few feet around the base of the pier.

68. Riprap foundations are, of course, very crude, and are not to be recommended, but it is nevertheless a fact that many railroad and highway bridges in the United States rest on such foundations, and that the percentage of failures has been small. In time, the currents of water wash gravel and soil into and fill the cavities between the masses of stone, thus forming comparatively solid and strong masses. Where rock is plentiful, these foundations are comparatively inexpensive, as they require no costly plant and only the crudest engineering skill.

69. Rock-Filled Timber Crib Foundations.—Timber cribs filled with riprap may be considered the next step in advance of simple riprap foundations. In these, a wooden house-like structure, without floor or roof, is built of round logs or squared timbers, floated into position, sunk, and filled with broken rock, the masonry resting on its top. The cribs are often built of round logs notched or flattened at the ends and intersections and secured together by drift bolts. The timber part of the structure must be permanently below the water; otherwise its top will soon decay. Sometimes the cribs are constructed with a close floor of timber or plank, but as a rule this is unnecessary.

Foundations of this kind have the advantage of cheapness where timber is plentiful and rock scarce, since, their sides being vertical, less rock is required. Besides, they offer less obstruction to navigation.

OTHER METHODS OF BUILDING FOUNDATIONS UNDER WATER

70. Open-Caisson Foundations.—The term *caisson*, as used in foundation work, is now commonly applied to any movable box-like structure having one open side and used to exclude the water from a foundation during its construction. In **open-caisson foundations**, the open side of the box is up; in pneumatic caissons, it is down. The open caisson is a water-tight box sunk in the water until its bottom rests

on the bed of the body of water, the masonry being built within and directly on its bottom. This description is general; in practice, many modifications of form and many methods of construction and operation have been employed.

The caisson is usually a very large box, whose horizontal dimensions are a few feet larger than the designed base of the masonry structure, whose bottom is strong enough to resist the upward pressure, and whose sides are able to resist the lateral pressure of the water when sunk to the desired depth. The floor is composed of one or more courses of timber laid closely side by side. If two or more courses are used, the timbers in alternate courses cross each other at right angles, the joints being calked or otherwise made water-tight. The plan for the construction of the sides depends largely on the depth and the pressure of water to be provided against.

The caisson may be built on land, launched, and floated to and securely moored into its correct position, and the laying of masonry begun on its floor. The caisson is thus soon sunk to its permanent position on the bottom, and the construction of the masonry continued until its top is above the water, when the sides of the caisson, being no longer needed, may be removed.

71. Combined Crib and Grillage Foundations. Where timber is plentiful and cheap, the submerged foundation may be constructed of it wholly, and the structure may be a combination of the grillage and the crib. The bottom courses may be built of timbers packed closely together to secure a large bearing area, and the top courses of similar construction to form a close platform for the masonry. In the intermediate courses, the timbers may be spaced with reference only to their strength to resist crushing due to the weight on them; and this part of the structure may resemble a grillage with large open spaces, which may or may not be filled with broken rock. Such a foundation is simply a large timber platform resting on the bottom, with its top approaching as near as desired to extreme low-water surface. It is

handled and built on in very much the same manner as described for the open caisson.

72. Preparation of the Bed or Subfoundation. The foundations described in Arts. **67, 69, 70,** and **71** have been spoken of as simply placed on the bed of the stream, which is assumed to be sufficiently level and firm for the purpose in its natural condition. This is, however, not always the case. The bottom may be uneven, or it may be composed of sand or silt, which, under the action of strong currents of water, may be undermined, allowing the foundation to be impaired or destroyed. Dredging is therefore often employed either to level the bottom or to excavate a pit into which the foundation may be sunk to such a depth below the bed of the stream that it will be, in a measure at least, secure from undermining. In some cases, crib and open-caisson foundations have been sunk to very considerable depths in sand and silt, by pumping or otherwise excavating the material from underneath the foundations through openings or shafts for the purpose, the foundation being forced downwards into the cavity by its own weight and that of the partly constructed masonry on it. In this way, crib foundations have been sunk to depths of as much as 80 feet below the bed of the stream.

73. Tubular Foundations.—Tubular or cylinder foundations, as described in Art. **35,** are frequently employed in water, the material being excavated from their interior by dredging. This method, or some modification of it, is practically the only one available where the depth that must be reached is very great, since the pneumatic process is not applicable to depths much exceeding 100 feet.

The foundations of the Hawkesbury bridge in Australia rest on a bed of compact gravel 185 feet below the surface of high water. They were sunk into place by a combination of the cylinder and open-caisson processes. The work could have been done by no other known method.

Single cylinders, in this process, are usually from 6 to 12 feet in diameter. A single one of these cylinders may

be capable of carrying the required load, but because of its small diameter it will lack lateral stability. For this reason, a group of two or more are sunk near each other, and their tops are connected with steel bracing. The cylinders, when sunk to the proper depth, are filled with concrete, or sometimes with gravel, iron caps being placed on and bolted to their tops, and the structure built on these caps.

74. Combination Pile and Cylinder Process.—A form of foundation used quite successfully for bridge piers is known as the **Cushing pile foundation.** Piles are first driven as close together as practicable, the outline of the group being a circle of the desired diameter. The piles are cut off just below low water, so that they will always be submerged. A cast-iron or steel cylinder of the required diameter is then lowered over and around the group of piles, and forced into the soft material of the bottom as far as possible, usually not less than 10 feet. The water is then pumped out and the vacant spaces around and among the piles are filled with concrete. The process is then completed as described in the last article.

The piles give the foundation bearing power and lateral stiffness, while the steel cylinder, in addition to adding to the strength of the structure, protects it from external injuries. Groups of such individual structures may be employed if necessary.

Tubes used in foundations have been spoken of as *cylinders*, and they are usually of that form; but they may be square or elliptic, and have been so made.

75. Pile Foundations Under Water.—Where the material of the bed of the body of water will permit it, pile foundations may be used. A system of properly located and spaced piles are driven, the length of the piles being preferably such that, when the driving is completed, their heads will be above the surface of the water. They are all then cut off to a level plane by a circular saw mounted on a vertical shaft. A timber caisson whose bottom is a platform made of two or more courses of timbers is then floated and

moored into exact position and sunk as heretofore described for open caissons, so as to rest on the tops of the piles. To prevent erosion around the piles and under the structure, riprap may be deposited around the caisson when the latter is in place.

76. Concrete Foundations Under Water.—Since hydraulic concrete sets or hardens perfectly under water, it is a most valuable and economical material for use in the construction of subaqueous foundations. With reasonable care, the freshly mixed concrete may be deposited without injury in water of great depth. If dumped loosely into the water and allowed to sink through it, the mortar will be washed out of the stone or gravel, and its usefulness will be destroyed. Several methods are employed to avoid this, the principal of which are as follows:

1. The concrete may be placed in partly filled, loosely woven cloth bags, the bags protecting the concrete while passing through the water. These bags, unopened, are placed by a diver in regular order in the foundation, being built up very much as blocks of stone in masonry. Sufficient mortar exudes through the material of the bags to cement them firmly together when in place.

2. Specially designed covered boxes or buckets having at their bottom a gate that may be opened by pulling a line, or that opens automatically when the bucket reaches the concrete already laid, are used. These buckets are handled by a derrick. When filled and closed, they are sunk through the water at the spot where the concrete is to be deposited, the gate is opened and the concrete thus deposited in quiet water in such a manner that the materials are not separated. This method is very expeditious and satisfactory and is largely employed.

3. A wood or metal tube, called a **chute,** long enough to reach from the floor of the foundation to a convenient distance above the surface of the water, is used. The concrete is filled into the top of this tube and thus conveyed to the point where it is to be deposited, the tube being moved

about as required to deposit the concrete regularly. If the tube is kept constantly full of concrete, water is excluded and the concrete may be thus put in place without injury.

FREEZING PROCESS FOR FOUNDATIONS

77. A process for sinking foundations through quicksand or semiliquid earth, called the **freezing process**, was invented by F. H. Poetsch, of Austria, and is therefore sometimes called the **Poetsch process**. It is substantially as follows:

The space to be excavated is surrounded by a series of vertical pipes sunk by the water-jet process. Each such pipe has a smaller pipe inside of it. The outer pipe is closed at its lower end, while the inner pipe is open at its lower end, and does not exactly reach to the bottom of the outer pipe. The several outer pipes are so connected that, when a refrigerating fluid is forced downwards through the inner pipe, it ascends through the space between the two pipes, is conducted thence to the next inner pipe, and is thus forced to circulate through the whole system of pipes. The refrigerating liquid is first reduced to a temperature much below the freezing point of water by apparatus similar to artificial-ice machines. The material around and between the pipes is, in time, frozen into a solid mass, which acts as the wall of a coffer dam in excluding the water or semiliquid mud from the space where the foundation is to be built. The material may then be excavated by ordinary methods. The process is necessarily slow and very expensive, but it has been successfully employed in a few cases where all other expedients had failed.

RETAINING WALLS

DEFINITIONS

1. Retaining walls are employed to hold the face of a bank in position, thus preventing it from sliding and caving. Specifically, a retaining wall is a wall built to hold in place a filling or backing of earth deposited behind it after the wall has been built.

2. The bottom of a retaining wall is called the **foot** of the wall. The front and back sides of the foot are called the **toe** and **heel**, respectively. In Fig. 1, *ef* is the foot of the wall *W*; *e* is the toe, and *f* the heel.

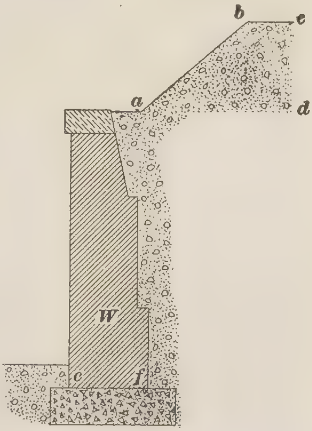


FIG. 1

3. A **vertical wall** is a wall in which both the front and the back are vertical.

4. Sea walls, or **wharf walls,** are masonry walls built along the water front. They sustain a pressure of earth from the back and take the wash of the water at the face. They are often required to support building walls, as in the case of ferry houses and wharf sheds, in which case they are subjected to considerable pressure from heavy cargoes.

5. A **face wall** is the same as a retaining wall, with the exception that the term is usually applied to a wall built

against the vertical face of natural earth formed by an excavation. The earth, being in its natural condition, is firm and solid, and is not so likely to slide and cave as filling.

6. Buttresses are piers of masonry built at intervals on the face of a wall, as *a*, *a*, Fig. 2. They are usually thicker at the bottom than at the top, and are employed to reinforce the wall by adding to its stability, and to prevent bulging. At present, buttresses are little used.

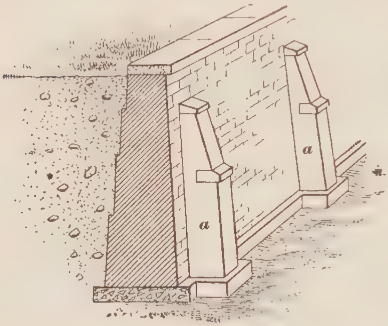


FIG. 2

7. Counterforts are projections or buttresses placed at the back of a wall, to supply additional weight in order to resist the overturning of the wall. Their utility is doubtful when built in connection with solid walls; but, when used with walls of reinforced concrete, they serve to tie a wide base to a thin-faced wall, so that the weight of the filling assists in maintaining the stability of the wall (see Fig. 3).

8. The terms **filling** and **backing** are understood to mean loose earth, gravel, or sand dumped to fill in an excavation back of a retaining wall.

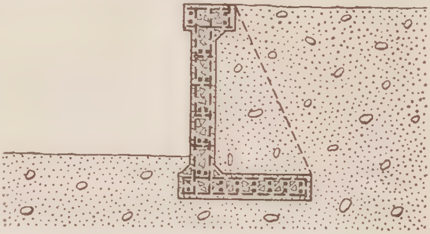


FIG. 3

9. Surcharge is that part of an earth embankment that is above the level of the top of the retaining wall. The earth shown in section at *a b c d*, Fig. 1, is the surcharge.

10. Land ties are tension bars or wire-rope guys secured to the back of a retaining wall and carried through the earth to masonry, large stones, plates, or timbers embedded in the

earth or filling, to which they are fastened. Any stone, plate, or timber used for this purpose is called a *dead man*. Land ties are never used in new work of good construction; they are a makeshift used principally to secure an old wall that shows signs of failure. Even for this purpose their usefulness is questionable.

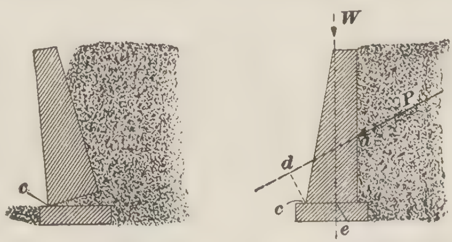
STABILITY AND DESIGN OF RETAINING WALLS

CAUSES OF FAILURE—COEFFICIENT OF FRICTION

11. Aside from the possible failure due to the settlement or upheaval of the foundation from softness of the supporting soil, or from frost or other cause (and this should be considered as failure of the foundation rather than of the wall itself), retaining walls may fail in three ways; namely, by *overturning*, by *bulging* in a vertical plane, or by *sliding* laterally.

12. Moment of Stability.—The **moment of stability** of a retaining wall is the moment of the weight of the wall about its toe, and is obtained by multiplying the weight of the wall by the distance from the toe to a vertical line passing through the center of gravity of the wall.

13. Overturning.—The overturning of a retaining wall—the most likely cause of failure—occurs whenever the moment of the pressure of the earth about the toe is greater than the moment of stability of the wall. Fig. 4 (*a*) illustrates the tendency of the wall to turn about its toe *c*. Referring to



(*a*) FIG. 4 (*b*)

Fig. 4 (*b*), the pressure of the earth is represented by *P*, its direction being shown by the arrow, and its point of

application, or the location at which it acts against the wall, being denoted by a. The force P tends to turn the wall about the toe c. In order that this tendency may be counterbalanced by the weight W of the wall, the moment of P about the toe c must not exceed the moment of W about the same point; that is, $P \times cd$ must not be greater than $W \times ce$, and the greatest value that P can have, consistent with the stability of the wall, is that obtained from the equation $P \times cd = W \times ce$.

14. Bulging.—The bulging of a retaining wall, as shown in Fig. 5, will occur when the top and bottom of the wall are held in place, as, for instance, in a cellar wall or in the wall of an area supported at the top by arches abutting against the wall, as shown at a. Bulging is likely to occur when the wall is thin, the point of greatest bulge usually being at about one-third the height from the footing, as at b; it is also sometimes caused by dumping the filling against a *green wall*—that is, a wall in which the cement or mortar has not had time to set properly.

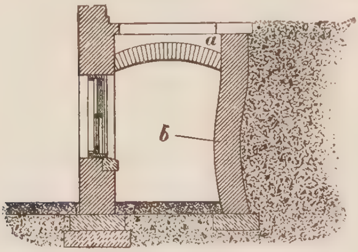


FIG. 5

A slight bulging in a retaining wall is not a serious matter, and need not be considered dangerous unless it exceeds $\frac{1}{4}$ inch for each foot in thickness of the wall at the point where the greatest bulging occurs.

15. Sliding occurs where the wall is sufficiently heavy and wide at the base to prevent overturning, but where, owing to the nature of the soil or the design of the footings, the friction between the base and either the footing or the soil is not sufficient to prevent a lateral movement of the wall. A wall is liable to fail by sliding in any horizontal joint, but, as the mortar has considerable adhesive value, any lateral movement that may occur is more liable to be between the masonry and the soil.

16. Coefficient of Friction and Angle of Repose. The subject of sliding friction is fully treated in *Analytic Statics*, Part 2, and here it will be sufficient to give the values of the coefficient of friction f and angle of friction or of repose Z, for the materials most commonly used in the construction of retaining walls. As explained in *Analytic Statics*, $f = \tan Z$.

Table I gives the coefficients of friction and angles of repose between different classes of masonry and between the soil and masonry for different conditions.

TABLE I

COEFFICIENTS OF FRICTION AND ANGLES OF REPOSE

Material	Coefficient of Friction	Angle of Repose Degrees
Fine-cut granite, on same, dry	.60	31
Fine-cut granite, on rough-pointed granite, dry	.65	33
Rough-pointed granite, on same, dry	.70	35
Well-dressed soft limestone, on same, dry	.75	37
Concrete blocks, on same, dry	.65	33
Concrete blocks, on fine-cut granite, dry	.60	31
Common brick, on same, dry	.65	33
Common brick, on well-dressed soft limestone, dry	.65	33
Common brick, on well-dressed hard limestone, dry	.60	31
Common brick, on same, with slightly damp mortar	.75	37
Hard brick, on same, with slightly damp mortar	.70	35
Hard limestone, on same, with slightly damp mortar	.65	33
Common brick, on same, with fresh mortar	.50	27
Well-dressed granite, on same, with fresh mortar	.50	27
Granite, roughly worked, on dry sand and gravel	.50 to .60	27 to 31
Granite, roughly worked, on wet sand	.35 to .45	19 to 24
Granite, roughly worked, on dry clay	.50	27
Granite, roughly worked, on moist clay	.35	19

The coefficients of friction of other classes of masonry on various soils may be taken the same as those given for granite in the table.

The coefficient of friction can hardly be given more exactly than within .05; slight differences in the conditions of the

materials or surfaces, which may not be perceptible to the ordinary observer, may make a very noticeable difference in the coefficient of friction. With mortar joints in masonry, the real coefficient is probably that between each of the courses of the stone and the mortar of the joint, with only a slight friction between the stones where they may be brought lightly in contact with one another at isolated points. The effect of moisture on the coefficient of friction is very well illustrated by a comparison of the coefficients for two surfaces of common brick in contact, as given in Table I: if dry, the coefficient is .65; if with slightly damp mortar, it is .75; while with fresh wet mortar, it drops to .5. As a rule, a slight dampness increases the friction, while saturation decreases the friction very materially. Clay is an exception to this rule: in it the friction is reduced by even a small amount of moisture.

TABLE II

COEFFICIENTS OF FRICTION, ANGLES OF REPOSE, AND WEIGHTS OF EARTHS

Material	Coefficient of Friction f	Angle of Repose Z Degrees	Weight Pounds per Cubic Foot
Mixed earth, dry .	.70	35	95
Mixed earth, damp	.80	39	115
Mixed earth, wet .	.40	22	115
Sand, dry	.65	33	110
Sand, wet	.05	3	125
Loam, dry	.70	35	75 to 100
Loam, wet	.50	27	90 to 120
Clay, dry	1.00	45	100
Clay, wet	.30	17	125

17. The angle of repose of loose materials corresponds to the angle of repose of the individual particles of which those materials are composed. Table II gives the coefficients of friction f and angles of repose Z of various kinds of earth,

with the weight per cubic foot. These quantities refer to the friction of each material on itself—that is, to the friction of the particles on one another. These elements are important factors in determining the stability of retaining walls. For ordinary calculations, the slope of repose is taken as $1\frac{1}{2}$ horizontal to 1 vertical, giving a coefficient of friction of .67, or an angle of 34°, with which a weight of backing of 100 pounds per cubic foot is used.

18. Where the footing of a retaining wall has a level bed, as in Fig. 6 (*a*), the resistance to sliding is equal to the weight of the wall (with any superincumbent load or force) multiplied by the coefficient of friction. In order to provide greater resistance to sliding than is obtained by the frictional resistance between the masonry and the soil or base with a level bed, the bottom surface of the footing is sometimes inclined inwards, as shown by *dc*, Fig. 6 (*b*). In this case, the resistance to sliding is equal to the component of the weight normal to the base *dc* multiplied by the coefficient of friction, plus the component of the weight parallel to the base.

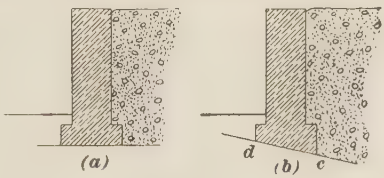


FIG. 6

If a wall is built with mortar joints, the backing should not be placed until the mortar has fully set, or there will be a much smaller resistance to sliding than may be expected after the mortar has set.

EXAMPLE 1.—A brick wall weighing 2,000 pounds rests on a level base of well-dressed soft limestone. What horizontal force will be necessary to slide the wall on the foundation, if the joint is without mortar and dry?

SOLUTION.—From Table I, the coefficient of friction for common brick on well-dressed soft limestone, dry, is found to be .65; multiplying this by the pressure on the bearing, 2,000 lb., gives 1,300 lb. as the horizontal pressure required to cause sliding. Ans.

EXAMPLE 2.—Assuming that the resultant horizontal thrust from the earth back of a granite retaining wall weighing 5,000 pounds per running foot and resting on a clay soil is 1,800 pounds in the same length

of wall, will the wall be secure against sliding, and what factor of safety will it possess when the clay is dry? How will the dampening of the clay modify the safety?

SOLUTION.—For a wall of this weight to be just on the point of sliding with this pressure at the back will require a coefficient of friction of $1,800 \div 5,000 = .36$. From Table I, the coefficient of granite on dry clay is found to be .5, and on moist clay .35. The factor of safety against sliding, is therefore, $.5 \div .36 = 1.4$, while the clay is dry. As .36 is greater than .35, the wall may slide if the clay becomes moist. Ans.

THEORY OF STABILITY

ASSUMPTIONS

19. Many theories relating to the stability of retaining walls have been evolved by eminent physicists. Although interesting, these theories have been proved by practice and experience to be unreliable. Nearly all of them are founded on three assumptions; namely, (1) that the line of rupture of the material composing the backing is a plane; (2) that the point of application of the resultant pressure on the back of the wall is the same as for water pressure; (3) that the direction of the thrust may be determined.

The first of these assumptions is known, by observation, to be false; while the others, though they seem reasonable, have not been conclusively proved.

20. In the **first assumption**, the earth at the back of the wall is considered as tending to slide along an oblique

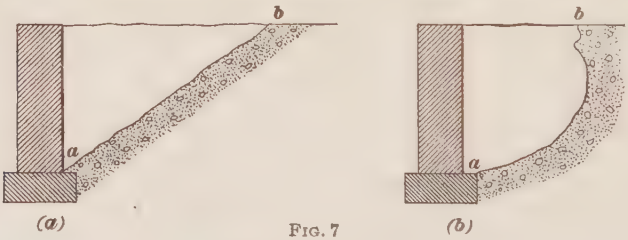


FIG. 7

plane $a\,b$, Fig. 7 (a). There is but one character of soil—clean dry sand—that will realize this theoretical condition;

all other soils possess considerable tenacity, and their line of rupture is a more or less irregular curve, such as *a b*, Fig. 7 (*b*).

The angle that the assumed plane of rupture makes with the horizontal, or the angle of repose, varies greatly with the different soils, and also with soils of the same nature containing different amounts of moisture. Damp sand or earth will stand when the slope has an inclination of 1 vertical to 1 horizontal. Roots also, serving as binders, have a large effect on the cohesiveness of the soil. The best practice in estimating the pressure of the filling on the back of a retaining wall is to consider the angle of repose for all earths and soils as 34°—that is, approximately $1\frac{1}{2}$ of horizontal run to 1 of rise. This is the natural slope of dry earth without roots, and as the pressure against a retaining wall is greater with dry material than with wet, provided that the latter does not possess such a degree of fluidity as to produce hydrostatic pressure, this angle is usually assumed.

21. The **second assumption**—that the point of application of the resultant pressure is the same as for water pressure—is equivalent to considering the earth backing as a fluid, devoid of tenacity and friction. Since the nature of earth varies so materially from that of water, this assumption is far from correct, though it is probably as nearly correct as any that can be made. Whatever error this assumption contains is on the side of safety.

22. The **third assumption**, regarding the direction of the thrust of the earth, varies with the several theories regarding the stability of retaining walls. In the theory advanced by Coulomb, the direction of the earth's pressure is assumed to be perpendicular to the face of the wall; this assumption disregards the friction of the earth against the face of the wall, and therefore cannot be correct. The physicist Weyrauch determines the angle that the line of pressure makes with the wall by a complicated trigonometric formula deduced from an elaborate theory that has little practical value; for, in opposition to all practical results, it

gives a greater pressure when the wall is inclined backwards toward the natural slope than when it is straight or leans forwards; it also neglects the vertical component of the earth's pressure, which certainly exists.

COULOMB'S THEORY

23. General Statement of the Theory.—The first theory (dating from the 18th century) for determining the stability of retaining walls, and the one on which most of the other theories are based, is known as **Coulomb's theory.** This is principally interesting as a study, and usually gives results that err on the side of safety, for walls that it demonstrates to be about to fall seem to be perfectly stable, and apparently possess a factor of safety of about 2. Coulomb assumed: (1) that the earth of an embankment held in place by a retaining wall tends to slide along a straight line or plane; (2) that the center of pressure of the earth acts at a

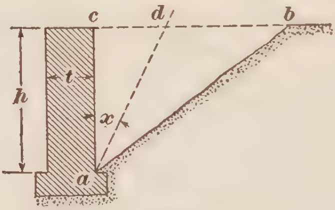


FIG. 8

point one-third of the height of the wall above the base; and (3) that the line of pressure acts perpendicularly to the back of the wall. From these assumptions, he determined, by reasoning and calculations, that the maximum pressure on a vertical wall is not due to the weight of the prism $c\,a\,b$, Fig. 8, but is caused by the wedge-shaped prism of earth $c\,a\,d$ included between the vertical back of the wall and the line $a\,d$ that bisects the angle $c\,a\,b$. The line $a\,b$ forms an angle with the horizontal equal to the angle of repose of the material. The line $a\,d$ is called the **slope of maximum pressure,** and the prism whose cross-section is represented by the triangle $c\,a\,d$ is called the **prism of maximum pressure.**

24. Formulas for Vertical Walls.—

Let P = total pressure, normal to the back of a retaining wall, per unit of length of wall;

W = total weight of triangular section of earth $c\,a\,d$, Fig. 8, between back of wall and slope of maximum pressure, for same length of wall;

W_1 = total weight of wall for same length;

w = weight of backing per unit of volume;

w_1 = weight of wall per unit of volume;

h = height $a\,c$ of wall;

t = thickness of wall, in the same units as h;

x = angle between back of wall and slope of maximum pressure, or angle $c\,a\,d$.

The volume of the prism $c\,a\,d$, whose altitude or depth is 1, is

$$\frac{a\,c \times c\,d}{2} \times 1 = \frac{h \times c\,d}{2} = \frac{h \times h \tan x}{2} = \frac{h^2 \tan x}{2}$$

Therefore,

$$W = \frac{w\,h^2 \tan x}{2} \qquad (1)$$

If the weight W is resolved into two components, one horizontal and one parallel to $a\,d$, the former is assumed to be the pressure P on the back of the wall. Therefore,

$$P = W \tan x$$

or, replacing the value of W from formula **1**,

$$P = \frac{w\,h^2 \tan^2 x}{2} \qquad (2)$$

Coulomb assumed that this pressure is equivalent to a single force of the same magnitude concentrated at a point one-third the height from the base $(= \frac{1}{3}h)$. Considering this as correct, and the value of P being as just determined, the thickness required for a vertical wall that it may be stable both as to overturning and sliding can be easily ascertained.

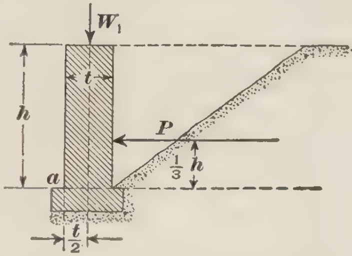


FIG. 9

25. Referring to Fig. 9, it being understood that a unit length of the wall is considered.

the moment of stability of the wall is $W_1 \times \frac{t}{2}$. The moment of P about a is $\frac{Ph}{3}$. Therefore, assuming P to be the maximum pressure consistent with stability (see Art. **13**),

$$W_1 \times \frac{t}{2} = \frac{Ph}{3}$$

But $W_1 = w_1 h t$; therefore,

$$w_1 h t \times \frac{t}{2} = \frac{Ph}{3};$$

whence
$$t = \sqrt{\frac{2P}{3w_1}} \qquad (1)$$

If the value of P, as given by formula **2**, Art. **24**, is substituted, the formula just given becomes, after reducing,

$$t = h \tan x \sqrt{\frac{w}{3w_1}} \qquad (2)$$

Since $c\,b\,a$, Fig. 8, is the angle of repose Z, we have

$$c\,a\,b = 90° - Z$$

and, therefore,

$$x = \tfrac{1}{2}(90° - Z) = 45° - \tfrac{1}{2}Z \qquad (3)$$

For a slope of $1\frac{1}{2}$ horizontal to 1 vertical, $\tan Z = \frac{2}{3}$; $Z = 33°\ 42'$; $x = 45° - 16°\ 51' = 28°\ 9'$; $\tan x = .535$; and, therefore,

$$t = .535\,h\sqrt{\frac{w}{3w_1}} \qquad (4)$$

26. The limit of stability as to sliding on the base occurs when

$$P = f\,W_1 = f\,w_1 h t, \qquad (1)$$

f representing the coefficient of friction between the base of the wall and its foundation or subfoundation. Substituting in (1) the value of P from formula **2**, Art. **24**,

$$\frac{w h^2 \tan^2 x}{2} = f\,w_1 h t;$$

whence
$$t = \frac{w h \tan^2 x}{2 f w_1} \qquad (1)$$

for the limit of stability as to sliding.

For a natural slope of $1\frac{1}{2}$ horizontal to 1 vertical, this reduces to

$$t = \frac{.14\, w\, h}{f\, w_1} \qquad (2)$$

The preceding formulas may be applied to any joint of the wall, by substituting for h the height of wall above the joint, and for f the coefficient of friction of the joints considered.

27. If the foundation is carried below the surface of the ground at the foot of the wall, the pressure of the earth in front will assist in opposing the tendency of the wall to slide on the foundation, and will also be of assistance in resisting overturning; but, unless the depth is greater than it is usually made, the increased resistance to overturning is unimportant. As the coefficient of friction for the masonry on the soil is frequently much less than that for two surfaces of masonry on each other, the assistance of the fill in front is often valuable in resisting sliding.

28. Wall With Battered Back.—For a wall with a battered back, following the assumptions made by Coulomb, the formulas should be modified by: (1) the increased overturning pressure due to the weight of the extra amount of backing $b\,c\,t$, Fig. 10, lying between the back of the wall and the vertical plane passing through its heel; (2) the decreased leverage of the overturning thrust, due to the fact that the line of action of the pressure, being perpendicular to the back of the wall, has a smaller arm ($q\,a$) than when the back is vertical ($p\,a$); (3) the reduction of the resisting weight of the wall in proportion to the thickness at the base ($b\,c\,t$ cut off from the wall); and (4) the decreased leverage, in proportion to the base, of the resisting weight of the wall due to its center of gravity k being thrown forwards ($a\,r$ is less than one-half the base).

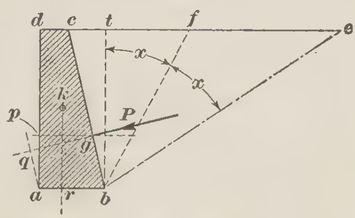


FIG. 10

All but the second of these modifications tend to increase, while this one decreases, the required thickness of the wall at its foot. The *average* thickness of wall required will be very nearly the same for a wall with a battered back as for a vertical wall.

29. Wall With Battered Face.—On the assumption of Coulomb's theory, battering the face of a wall modifies the formulas for overturning: (1) by reducing the weight of the wall in proportion to its base by cutting off $a_1 a d$, Fig. 11; and

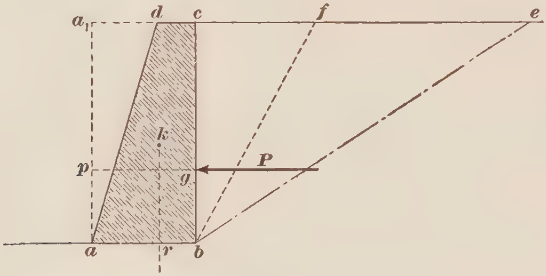


FIG. 11

(2) by increasing the leverage of this resistance to overturning ($a r$ is more than one-half the base). The net result is a small increase of base required, but a decided decrease of average thickness as compared with a vertical wall.

30. Remarks.—Coulomb's theory is unsatisfactory, because of the incorrect assumptions; because of the neglect of the friction on the back of the wall and along the plane of slope of maximum pressure, and because of the vertical pressure of the earth on the back of the wall when the latter is built with a slope or with steps. The theory is further at fault for a surcharged wall, in that in this case the slope of maximum pressure will no longer be midway between the vertical and the slope of repose. Though this theory is very imperfect, it has been given here because nearly all the theories that have been advanced for retaining walls are based on it; and, as the best theories are simply amplifications of Coulomb's, they can perhaps be better explained and understood by studying the latter first. On account of the fact that

the theory, as originally proposed, is no longer used, it is scarcely worth while to give here the modifications that must be introduced in the formulas derived for vertical walls to make them applicable to battered walls. For the same reason, the treatment of surcharged walls by Coulomb's theory is omitted entirely.

EXAMPLE.—A vertical retaining wall of rubble masonry, laid in cement mortar and weighing 150 pounds per cubic foot, is to be built with the backing level with the top of the wall, and 16 feet above the top of the foundation, which is at the level of the ground below. What thickness of wall will be required, by Coulomb's theory, if the backing weighs 100 pounds per cubic foot and has a slope of repose of $1\frac{1}{2}$ horizontal to 1 vertical, and the coefficient of friction of the masonry on the foundation is .65?

SOLUTION.—To apply formula **4**, Art. **25**, we have $h = 16$, $w = 100$, and $w_1 = 150$. Therefore,

$$t = .535 \times 16\sqrt{\frac{100}{3 \times 150}} = 4.04 \text{ ft.}$$

Substituting the same values, and also that of f in formula **2**, Art. **26**,

$$t = \frac{.14 \times 100 \times 16}{.65 \times 150} = 2.3 \text{ ft.}$$

As this thickness is less than that required for stability against overturning, 4.04 feet is the proper thickness to use.

MOSELEY'S THEORY

31. General Statement of the Theory.—Moseley's theory of retaining walls is a modification of, and a great improvement on, Coulomb's theory. It takes account of the friction of the earth on the back of the wall, and also, if the back of the wall is battered, of the vertical pressure on the horizontal projection of the batter. When the back of the wall is battered or stepped, the effect of the vertical pressure of the earth resting on the slope or the steps far exceeds the overturning pressure due to the extra amount of backing.

The effect of friction is a very important factor, as it about doubles the stability of a wall under ordinary conditions, as compared with what it would be if there were no friction.

32. In Moseley's theory, the *horizontal component* X, Fig. 12, of the pressure of the filling on the wall is assumed to be as in Coulomb's theory; that is (formula **2**, Art. **24**, and formula **3**, Art. **25**),

$$X = \frac{w\,h^2 \tan^2 (45° - \frac{1}{2} Z)}{2}$$

The *total pressure* P is the resultant of the horizontal pressure X and the friction Y between the wall and the filling. This friction is equal to $f_1 X$, where f_1 is the coefficient of friction between the material of the wall and that of the

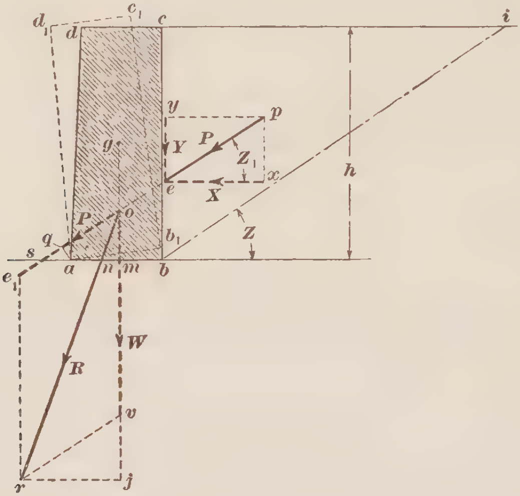


Fig. 12

filling; and the angle $p\,e\,x$ is equal to the angle of friction Z_1, whose tangent is f_1 (see *Analytic Statics*, Part 2). The point of application of P is assumed, as in Coulomb's theory, to be such that $b\,e = \frac{1}{3}\,b\,c = \frac{1}{3}\,h$.

The effect of friction will be better understood by considering the condition of critical equilibrium. When the wall is in that condition, the slightest increase of pressure at the back will cause overturning. In overturning about the toe a, the back of the wall rises and the wall moves forwards, as indicated by the dotted lines $a\,b_1\,c_1\,d_1$, the motion being first vertical, and gradually changing to a forward motion as the

wall revolves about the toe. As the wall starts to rise, the backing starts to fall and fill the gap caused by this motion, and the pressure of the earth on the back of the wall produces a friction, represented by $y\,e$, whose magnitude is proportional to the normal pressure X and to the coefficient of friction f_1. Even though there is actually no motion, the power of resistance is still present to prevent overturning; it modifies the effect of the pressure of the fill on the back of the wall, and increases the stability of the wall.

The coefficient of friction f_1 of the earth against the masonry is generally taken as .65 for dry earth; this corresponds to a slope of nearly $1\frac{1}{2}$ to 1.

33. Pressure on Base of Wall.—The total pressure R, Fig. 12, acting on the base of the wall is the resultant of the pressure P and the weight W of the wall. Its magnitude and line of action are determined by the parallelogram $o\,e_1\,r\,v$, in which $o\,e_1 = P$ and $o\,v = W$, the point o being the intersection of the line of action of P with a vertical through the center of gravity g of the wall. To find R by calculation, the sides $o\,e_1$ $(= P)$ and $e_1\,r$ $(= W)$ of the triangle $r\,e_1\,o$ are known; also, angle $o\,e_1\,r = 180° - e_1\,o\,v = 180° - e\,p\,x = 180° - (90° - p\,e\,x) = 90° + p\,e\,x = 90° + Z_1$.

34. If both the wall and the foundation were absolutely incompressible and incapable of fracture or crushing, the wall would be safe from overturning if the point n where the line of action of R meets the base came anywhere inside the base of the wall; and, theoretically, the pressure P could be increased until n coincided with a—that is, until the line of action of the resultant pressure R passed through the toe a. But, as explained in *Foundations*, Part 1, practical considerations require that, under ordinary conditions, the point n should fall within the middle third of the base of the wall. The theory of pressure on foundations is fully treated in that Section, which should be referred to for further particulars. It must be stated, however, that the distance $a\,n$ may safely be reduced somewhat from one-third to even one-fifth the width of the base, if the foundation is perfectly

rigid and the masonry of the best. This will give a maximum intensity of pressure on the foundation at a $3\frac{1}{3}$ times the intensity there would be if the center of pressure were at the center of the base.

35. Stability Against Sliding.—The total pressure R on the base may be resolved into a vertical component oj $(= W + Y)$ and a horizontal thrust jr $(= X)$ tending to produce sliding on the base. This thrust must not exceed the product of the normal pressure oj and the coefficient of friction between the wall and its foundation; otherwise expressed, the angle roj must not exceed the angle of friction between the wall and its foundation, unless some external means, such as earth placed in front of the wall at the base, is employed to strengthen the wall against sliding.

Ordinarily, the friction of the back filling is disregarded in determining the resistance to sliding. It is, however, advisable to take it into account, for though latent when there is no motion of the wall, the instant that the wall begins to move, or is about to do so, whether by overturning or by sliding, the filling begins to slide—or is ready to do so—down the back of the wall, and brings the friction into action. The neglect of this factor of stability against sliding is the more readily yielded to because usually the thickness of wall required for stability against overturning gives ample weight to resist sliding, and the added help of the filling in front of the foundation, required on account of frost and other surface influences, is generally sufficient to make up for the neglected friction of the filling.

EXAMPLE.—Taking the example given in Art. **30**, with a wall 4 feet thick, and assuming the wall and foundation to be perfectly rigid and the coefficient of friction of the masonry on the foundation and on the backing = .65: (*a*) what are the factors of safety against overturning and sliding? (*b*) where is the point of application of the resultant pressure R, Fig. 12?

SOLUTION.—(*a*) In this case (see Art. **30**), tan x = .535; h = 16, and w = 100; therefore, referring to Fig. 12 (see formula **2**, Art. **24**),

$$X = \frac{100 \times 16^2 \times .535^2}{2} = 3{,}660 \text{ lb., nearly.}$$

Since $f_1 = .65$, we have

$$Y = .65\ X = .65 \times 3{,}660 = 2{,}380 \text{ lb., nearly}$$

Also, $\tan Z_1 = .65$, $Z_1 = 33°\ 1'$, and

$$P = \frac{X}{\cos Z_1} = \frac{3{,}660}{\cos 33°\ 1'} = 4{,}365 \text{ lb.}$$

To find the lever arm $a q$, Fig. 12, produce $p e$ to its intersection s with $b a$ produced. Then,

$$b s = b e \cot e s b = b e \cot p e x = \tfrac{1}{3}\, b c \cot Z_1 = \frac{b c}{3 \tan Z_1} = \frac{16}{3 \times .65} = 8.2 \text{ ft.}$$

Also $\quad a s = b s - b a = 8.2 - 4 = 4.2$ ft.;

$$a q = a s \sin e s b = a s \sin Z_1 = 4.2 \sin 33°\ 1' = 2.29 \text{ ft.}$$

The moment of P about a is, therefore,

$$4{,}365 \times 2.29 = 9{,}996 \text{ ft.-lb.}$$

The weight of the wall, per foot of length, is $16 \times 4 \times 150 = 9{,}600$ lb., and the moment of stability of the wall is

$$9{,}600 \times m a = 9{,}600 \times 2 = 19{,}200 \text{ ft.-lb.}$$

Dividing this by the moment of P, the factor of safety against overturning is

$$19{,}200 \div 9{,}996 = 1.92, \text{ nearly. Ans.}$$

The force tending to cause sliding is the horizontal thrust X, or 3,660 lb. The frictional resistance that the wall can offer is

$$o j \times f_1 = (W + Y) \times f_1 = (9{,}600 + 2{,}380) \times .65 = 7{,}790 \text{ lb.}$$

The factor of safety against sliding, is, therefore,

$$7{,}790 \div 3{,}660 = 2.13, \text{ nearly. Ans.}$$

(*b*) In the triangle $o m n$,

$$o m = m s \tan o s m = (b s - b m) \tan Z_1 = (8.2 - 2) \times .65 = 4.03 \text{ ft.}$$

In the triangle $o r j$,

$$\tan r o j = \frac{r j}{o j} = \frac{X}{W + Y}$$

In the triangle $o m n$,

$$m n = o m \tan r o j = 4.03 \times \frac{X}{W + Y} = 4.03 \times \frac{3{,}660}{9{,}600 + 2{,}380} = 1.23 \text{ ft.}$$

Ans.

36. Remarks.—In actual practice, it is customary to find the magnitude and direction of the maximum pressure graphically, instead of by calculation. As the weight of the backing, and the angle of friction, as well as the strength, cohesion, and compressibility of the materials, are so imperfectly known, the results may be determined with sufficient accuracy by graphic instead of analytic methods.

37. It is to be noted that, in the foregoing example, the wall is a trifle thinner than would be required by Coulomb's theory with no factor of safety; yet, by Moseley's theory,

the wall has a factor of safety of nearly 2. The difference is due to the introduction of the friction on the back of the wall.

38. Battered Back.—To ascertain the resultant of the pressure of the bank and the friction it develops when the back of the wall is battered, the angle of friction should be

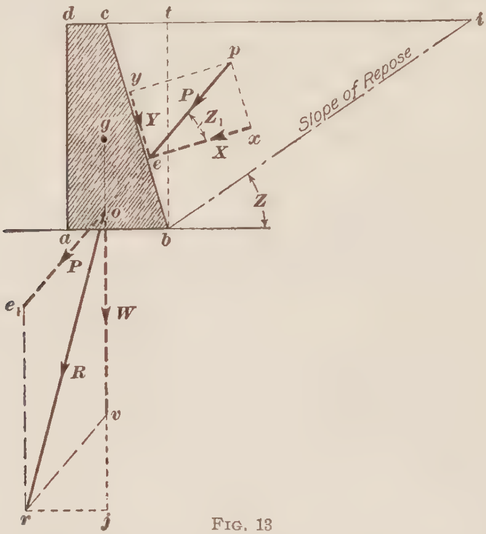


FIG. 13

laid off from a line normal to the back, as shown in Fig. 13. This will make allowance for the greater overturning pressure of the backing (due to the larger wedge of earth between the back of the wall and the slope of maximum pressure, and also to the greater wedging angle between these two planes) and for the extra resistance to overturning due to the weight of earth acting vertically on the horizontal projection of the batter of the back. The net result of this deflection of the line of thrust is to reduce the overturning moment to be resisted by the weight of the wall,

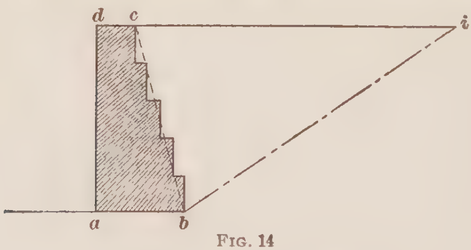


FIG. 14

so that, with the same average thickness of wall, and the face remaining plumb, the stability will be increased. If the batter of the back is sufficient, the line of action of P will cut the base, and there will be no overturning moment. The batter of the back also reduces the weight and leverage of the wall.

If the back of the wall is stepped, or offset, as in Fig. 14, the line of thrust will be the same as though the back were battered to the plane bc.

39. Surcharged Walls.—With a surcharged wall, as in Fig. 15, the weight of all the earth $c\,b\,k_1$ lying between the slope of maximum pressure, the back of the wall and the top of the surcharged slope should be employed instead of the smaller quantity $c\,b\,k$ that would be used if the filling were leveled off even with the top of the wall. According to this theory, if the surcharge is carried higher than the point k_1, where the slope of maximum pressure intersects the slope of the top of the surcharge, as to u, the pressure of the filling on the back of wall is not increased thereby (see Art. 48); but, if the top of surcharge is lower than k_1, as at u_1, the weight of earth in the prism $l\,b\,c\,u_1$ should be used.

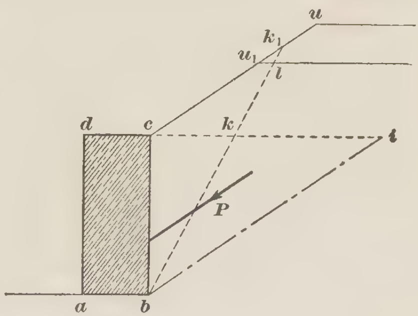


Fig. 15

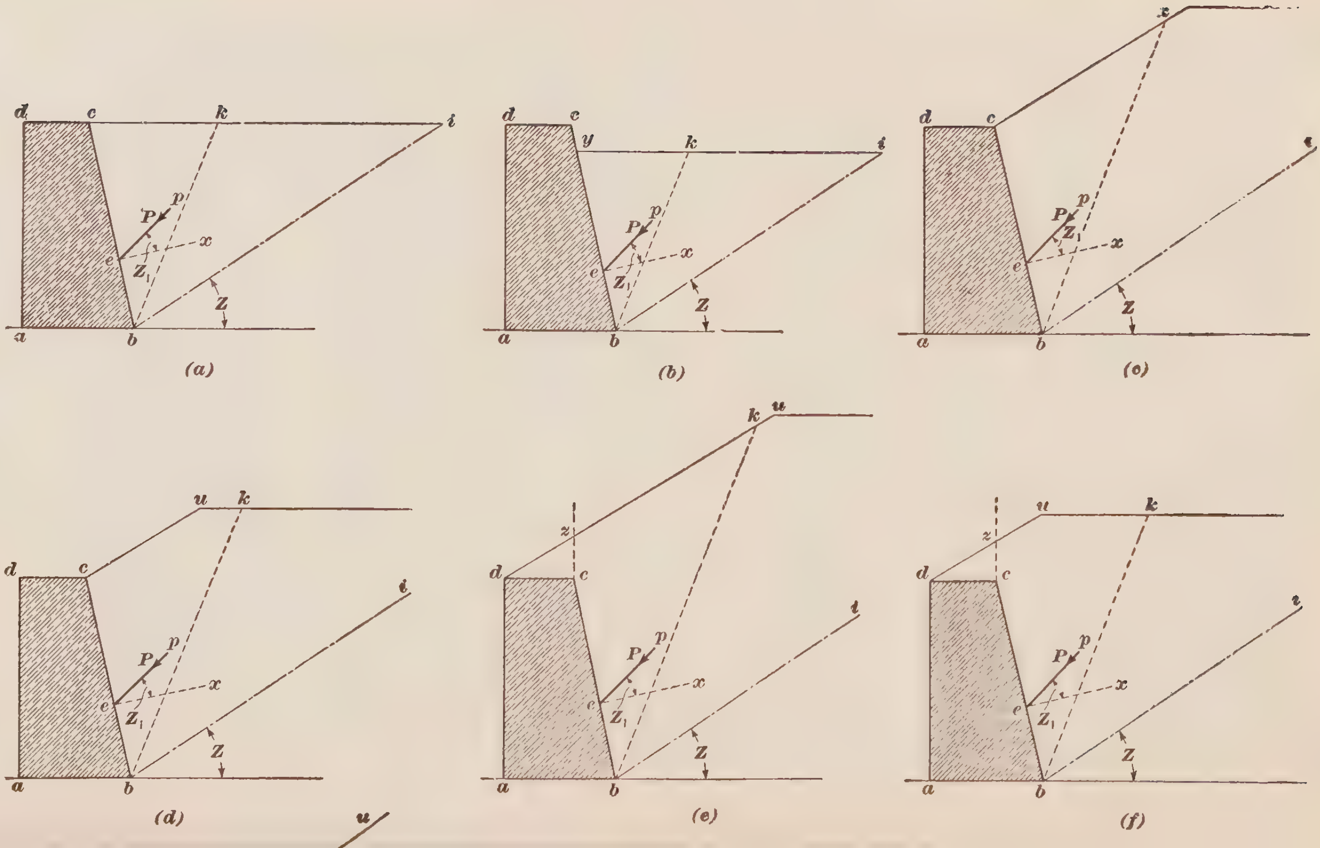
If the surcharge is allowed to run over on the top of the wall, as in Fig. 16 (e), (f), and (g), only the part of the surcharge lying behind a vertical plane $c\,z$ carried up from the back edge of the top of the wall should be used in determining the pressure of the filling; while the weight of the earth lying in front of this plane should be added to the weight of the wall, to find the resistance offered to overturning. In all cases, *the weight of earth to be used should be the weight of the earth lying between the slope of maximum pressure* $b\,k$,

Figs. 15 and 16, the upper surface of the soil, the back of the wall, and a vertical plane c z, Fig. 16, extended upwards from the upper edge of the back. This general rule applies whether the earth is leveled off with the top of the wall, as *c k i*, Fig. 16 (*a*), or at a lower level, as *y k i*, Fig. 16 (*b*); whether the wall is surcharged with the toe of the slope lying at the back of the top of the wall, as *c k b* in (*c*) and *c u k b* in (*d*); surcharged with the slope running over the top of the wall, as *c z k b* in (*e*), *c z u k b* in (*f*), and *c z k b* in (*g*); surcharged with the toe of the slope back from, or below, the top of the wall, as *c y k b* in (*h*) or *y k b* in (*i*); or whether the top of the earth is lower behind the wall than its top, as *c k b* in (*j*), *y k b* in (*k*), and *c y k b* in (*l*).

40. The direction of the pressure is unchanged by the position of the top of the backing, being governed by the batter (or equivalent offsetting) of the back of the wall and by the angle of friction between the backing and the masonry.

41. The point of application of the resultant pressure is the point where a line parallel to the slope of maximum pressure, and passing through the center of gravity of the mass of earth causing this maximum pressure, cuts the back of the wall. If the cross-section of this mass of earth is triangular, with its apex at the top of the back of the wall—whether from this point the surface of the backing is level, as at Fig. 16 (*a*), or rising, as at (*c*), or falling, as at (*j*)—this point of application is at one-third the height of the wall, for the center of gravity of every triangle is at one-third its altitude measured from any side as a base; and a line passing through this point and parallel to the assumed base will cut each of the other sides at one-third its length from this base.

If the cross-section is triangular, with its apex at the back of the wall below the top, as at (*b*), (*i*), or (*k*), the point of application will, of course, be lowered to one-third the height of the wall to this apex. If the cross-section has more than three sides—whether this is caused by the top of surcharge lying below the intersection of the slope of



(a) (b) (c)

(d) (e) (f)

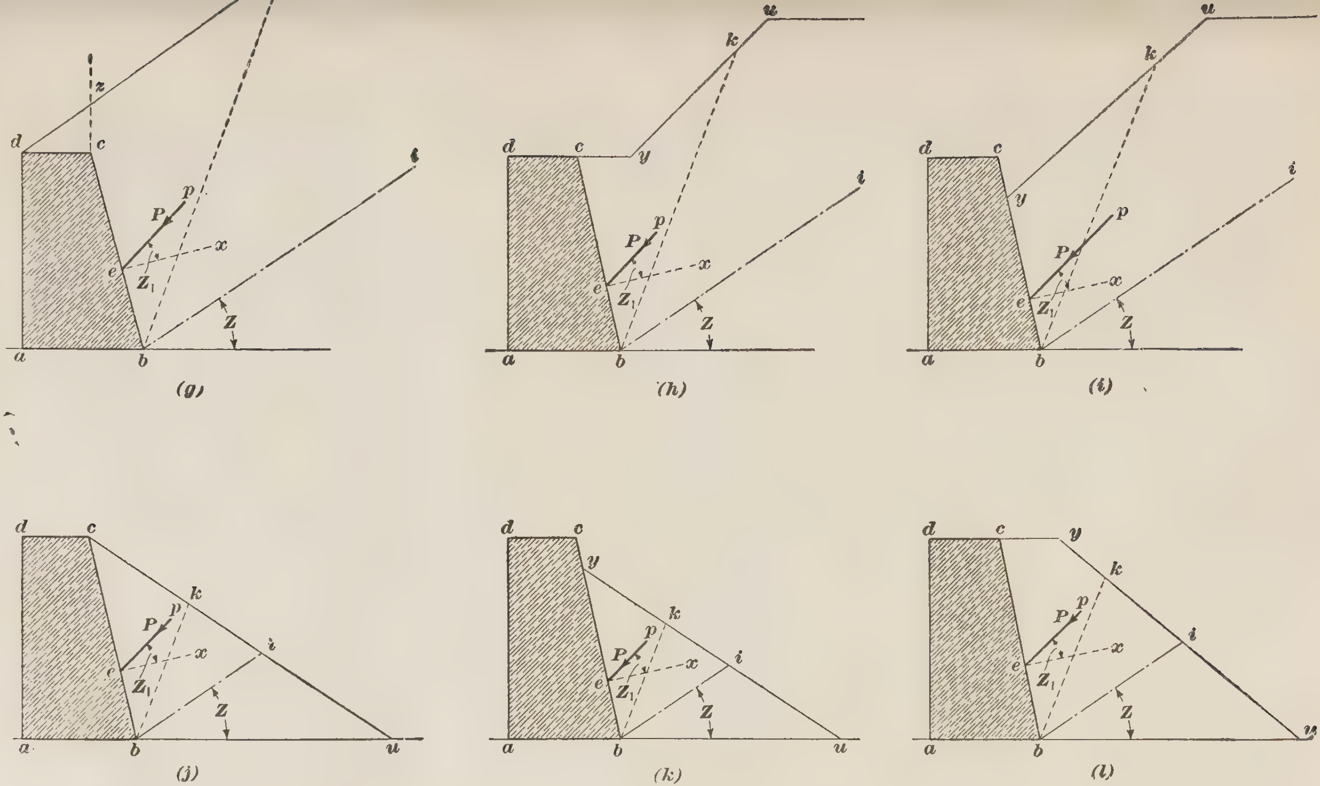


Fig. 16

its face with the slope of maximum pressure, as at (*d*) and (*f*), or because the surcharge runs over the top of the wall, as at (*e*), (*f*), and (*g*), or by the backing sloping down from a point behind the back of the wall, as at (*l*)—the point of application of the resultant pressure will be raised, because the center of gravity of the mass of backing causing the maximum pressure will lie farther from the slope of maximum pressure. The one possible exception to this rule, where the cross-section has a perimeter of more than three sides, is when it contains a reentrant angle, as *y*, Fig. 16 (*h*).

It will be observed that the effect of lowering the level of the top of a surcharge, and so reducing the amount of backing—as at (*d*) compared with (*c*), and at (*f*) compared with (*e*)—or the removal of some of the backing when normally level—as at (*l*) compared with (*a*)—raises the point of application of the pressure, and so increases its leverage; but, as the amount of the pressure is at the same time reduced to more than offset this increase of leverage, the moment of the overturning pressure is reduced.

42. Battered Face.—According to Moseley's theory, the use of a battered face on a retaining wall has no effect on the overturning moment, but modifies the resistance offered by the wall in the same manner as explained in Art. **29** in connection with Coulomb's theory, increasing slightly the required base, but decreasing materially the average thickness and total quantity of masonry.

43. General Method of Procedure.—In what follows, the preceding principles will be combined into a general statement indicating the successive steps that should be taken in the analysis of a retaining wall according to Moseley's theory.

Fig. 17 represents a surcharged wall *a b c d* with the toe of the surcharge lying at the outer face *d* of the top of the wall; the description and directions, however, will apply equally well, in substance, to other conditions. The wall may be stepped, as indicated by the dotted lines, but this does not alter the principles involved.

First lay off from the heel of the wall the slope of repose $b\,i$ of the material of which the backing is made. Usually, the slope of $b\,i$ is assumed as $1\frac{1}{2}$ horizontal to 1 vertical, as this is approximately the worst slope to be expected with the materials commonly used for filling. The slope of maximum pressure $b\,k$ for any material and condition lies midway between the slope of repose $b\,i$ and the vertical plane $b\,t$ passing through the heel of the wall; that is, $b\,k$ bisects $t\,b\,i$.

Considering, now, a section of wall and backing contained in one unit of length measured parallel to the line of the wall, ascertain the weight and center of gravity g_1 of the backing lying between the slope of maximum pressure and the back $b\,c$ of the wall, with an extension $c\,z$ in a vertical plane above the top of the back of wall, to the slope of the surcharged filling; that is, of the mass represented in cross-section by the polygon $k\,b\,c\,z\,u\,k$. To this mass should be added the weight of any load that may rest on the filling. From the center of gravity of this portion of the backing (with its superimposed load, if any), draw a line $g_1 e$ parallel to the slope $k\,b$ of maximum pressure, until it intersects the back of the wall at e, which represents the center of pressure of the backing—that is, the point of application of the resultant pressure. Through e draw $e\,x$ perpendicular to the back of the wall (or to the plane passing through the back of the wall at its top and bottom, if the back of the wall is stepped), and also a vertical line, on which scale off a

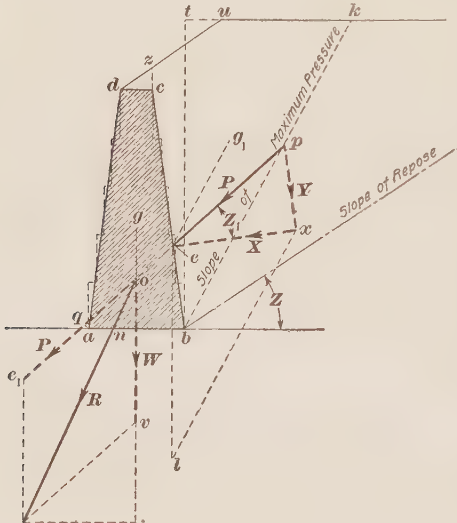


FIG. 17

length el to represent the weight of the filling, with its superimposed load, if there is any. Draw lx parallel to bk. Then, ex will represent the component X of the pressure P perpendicular to the back of the wall. Draw xp parallel to the back of the wall, and ep making with ex an angle Z_1 equal to the angle of friction between the filling and the back of the wall (usually taken as 33°, which corresponds to a slope of $1\frac{1}{2}$ to 1). Then, px will represent the magnitude and direction of the friction Y that may be developed by the pressure ex of the filling on the back of the wall; and pe will represent the resultant pressure P. Producing pe and dropping a perpendicular qa to it from the toe a of the wall, the lever arm of P with respect to a is determined. The product of pe (in units of weight or pressure) and qa (linear units) represents the overturning moment.

Now find the weight W and the center of gravity g of the wall $abcd$, together with the small amount of filling dcz resting on its top (and also of any other load or force resting on the wall, if there is any), and from this center of gravity draw the vertical gj. From the point o where this vertical meets the line of action of P, lay off $ov = W$, and $oe_1 = P$; complete the parallelogram e_1ovr, and draw the diagonal or, which represents the resultant pressure R on the base or foundation of the wall; this resultant intersects the base at n. Under ordinary conditions, an should not be less than $\frac{1}{3}ab$.

The resultant total pressure or may be resolved into a vertical component oj and a horizontal component rj; the latter thrust must not exceed the product of the former and the coefficient of friction of the wall on the base (or the angle roj must not exceed the angle of this friction), unless there is some other force that helps to resist the tendency to slide.

EMPIRICAL RULES

44. As previously stated, none of the mathematical theories of the equilibrium and stability of retaining walls is altogether satisfactory, because of the many and important elements of uncertainty entering into them. But, by the use of the best theories available, combined with the results of experience and observation, very simple rules have been formulated that probably are a more correct guide than even the most perfect theory yet advanced. A thorough understanding of these theories is, however, very important, both because they are often used or referred to, and also because they give a general idea at least of the conditions that should be taken into account.

Probably the best series of empirical rules is that presented by John C. Trautwine in his well-known *Civil Engineers' Pocketbook*. These rules are given, with slight modifications, in the following articles.

45. Vertical Walls.—For a vertical wall resting on a foundation of masonry suitably enlarged for a proper distribution of the load on the soil, with the top of the fill leveled off at the top of the wall, and with the backing deposited loosely, as when dumped from carts, the ratio of the thickness to the height of the wall should be .35 for a wall of cut stone, or of first-class large-ranged rubble, in mortar, or of concrete; .4 for a wall of good common rubble or brick, in mortar; and .5 for a wall of dry rubble.

If the backing is deposited in layers well compacted, the thickness may be slightly reduced, without sacrificing the stability of the wall. It is not, however, customary to reduce the thickness on this account; for, although specifications frequently call for the filling to be so deposited and compacted, this is rarely done, and an engineer attempting to enforce such specifications will be likely to receive more abuse than satisfaction.

46. Battered or Stepped Back.—For a wall with a battered or stepped back, Trautwine recommends using the

same *average* thickness as for a vertical wall, increasing the base by the same amount that the top width is decreased. As seen in the discussion of Moseley's theory, this will give a small increase in the stability over that obtained from the vertical wall, while the volume of masonry remains the same. The reduction in average thickness can only be very slight without decreasing the stability, so that it is ordinarily disregarded.

47. Battered Face. A wall with a battered face may be made to give the same stability with a materially smaller volume and average thickness that would be required if a vertical wall were used.

For equal stability, a wall of triangular cross-section with a vertical back and battered face should have a base 1.225 times as great as the thickness of a vertical wall, as may be seen by reference to Fig. 18, in which $a\,b\,c\,d$ represents a vertical retaining wall, and $a_1\,b\,c$ a wall of triangular section having the same height and moment of stability: g and g_1 represent their respective centers of gravity, and $m\,a$ and $m_1\,a_1$ the lever arms of their weights about their respective toes.

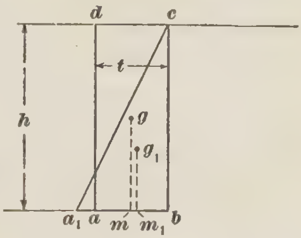


FIG. 18

Let h = height of wall;

t = thickness of vertical wall;

b = width of base of triangular wall;

w = weight of masonry per unit of volume.

Then, in a portion of the wall one unit in length, the weight of the vertical wall $a\,b\,c\,d$ will be $h\,t\,w$, and its moment of stability,

$$h\,t\,w \times \frac{t}{2} = \frac{h\,t^2\,w}{2}$$

For the same length, the weight of the triangular wall will be $\frac{b}{2}h\,w$, and its moment of stability,

$$\frac{b}{2}h\,w \times m_1\,a_1 = \frac{b}{2}h\,w \times \tfrac{2}{3}b = \frac{h\,b^2\,w}{3}$$

In order that these two walls may have the same degree of stability, we must have

$$\frac{h\,t^2\,w}{2} = \frac{h\,b^2\,w}{3};$$

whence $$b = t\sqrt{\tfrac{3}{2}} = 1.225\,t$$

The amount of masonry in the triangular wall of equal stability against overturning will therefore be only .6125 times that of the vertical wall. Such a wall will, however, have serious objections, as will be explained further on.

In a similar manner, the dimensions of a wall having a vertical back, a given batter of face, and a given stability may be computed.

48. Surcharged Walls.—As indicated in Art. **39**, the pressure on the back of a surcharged wall is greater than if the backing were level with the top of the wall, and the thickness of the wall must be correspondingly greater.

When the surcharge runs over the top of the wall—as in Fig. 17, and in Fig. 16 (*e*), (*f*), and (*g*)—there is a slight increase in the weight resisting overturning by the addition of the triangle of earth $d\,c\,z$, Fig. 17, as well as the larger increase in the weight of the wedge of backing pressing against the back of the wall. For a height of surcharge less than about a quarter of the height of the wall, the additional weight of the filling resting on the top of the wall will offset the extra weight of the overturning wedge; but, as the height of the surcharge increases with the slope running over the top of the wall, the overturning pressure increases rapidly, while the increased resistance due to the earth resting on the top of the wall changes only slightly with the increase in thickness of the wall. Thus, it is readily seen that, if the surcharge is large and is allowed to run over the top, the thickness of wall required will be greater than if the surcharge slope started at the back of the wall. Table III shows the proper ratios of thickness to height for vertical walls with various amounts of surcharge. After ascertaining the thickness of the vertical wall that would be required for restraining a surcharge bank, the form of the wall may

be altered to give a battered face or back, or both, in the same way as though the top of the backing were level with the top of the wall (see Arts. **46** and **47**).

It will be noticed that Table III calls for an increase in the thickness of the wall for the increased height of the surcharge, far beyond the .55 times the height of the wall at

TABLE III

SURCHARGED VERTICAL WALLS—RATIO OF THICKNESS TO HEIGHT

Ratio of Surcharge to Height of Wall	Toe of Slope at Back of Wall			Toe of Slope at Front of Wall		
	Cut Stone	Mortar, Rubble, or Brick	Dry Rubble	Cut Stone	Mortar, Rubble, or Brick	Dry Rubble
.0	.35	.40	.50	.35	.40	.50
.1	.42	.47	.57	.42	.47	.57
.2	.46	.51	.61	.46	.51	.61
.3	.49	.54	.64	.49	.55	.66
.4	.51	.56	.66	.53	.60	.72
.5	.52	.57	.67	.58	.65	.79
.6	.54	.59	.69	.62	.70	.85
.7	.55	.60	.70	.65	.74	.91
.8	.56	.61	.71	.67	.77	.96
.9	.57	.62	.72	.69	.80	1.00
1.0	.58	.63	.73	.71	.82	1.04
2.0	.62	.67	.77	.81	.96	1.26
3.0	.63	.68	.78	.85	1.02	1.35
5.0	.64	.69	.79	.88	1.07	1.44
25.0	.68	.73	.83	.92	1.11	1.50

which the "plane of maximum pressure" will usually cut the plane of natural slope with the toe of surcharge at the back of the top of the wall. This is because the results of experiment and experience show that the pressure actually does increase beyond the limit indicated by the best theories that have yet been formulated. Plainly, some important factor is omitted in these theories.

SETTLEMENT OR CRUSHING OF THE TOE

49. It should be noted that even the hardest stone is compressible and may be crushed, and that the subfoundation is generally even less rigid and strong. There is, consequently, a greater or less yielding of wall and subfoundation caused by the pressure of the backing; and the greater the pressure, the greater will be the yielding. As the pressure on the base of a retaining wall is greater at the toe than at the heel, this yielding is also greatest at the toe. With a well-designed wall resting on a firm subfoundation, this yielding is so slight as to be negligible, and generally hardly perceptible even with the most careful observations; but, when the subfoundation is soft and the center of pressure lies very near the toe of the wall, the yielding may become of great importance.

50. The destruction of the toe of a retaining wall decreases the lever arm of the weight, and, therefore, the moment of stability of the wall. It increases also the intensity of pressure on the subfoundation, by reducing the width of base.

51. By the settling of the toe, the wall leans forwards, thereby throwing its center of gravity more nearly over the toe, thus further reducing the lever arm of the weight and the moment of stability of the wall. This also increases the pressure on the toe, tending to increase (and more rapidly) the settlement there, and speedily leading to the downfall of the wall.

52. The settlement at the toe and the resulting leaning forwards of the wall increase the pressure of the backing, by increasing the amount and weight of the filling lying between the back of the wall and the slope of maximum pressure, and by enlarging the wedging angle through which the pressure tends to throw the wall forwards. As the wall yields to this increased pressure, the pressure still further increases.

53. By the settlement or crushing of the toe, the wall leans forwards; the beds become inclined, and the tendency of the wall to slide as a whole on its base, and of the different beds to slide on each other, is increased. After a sliding motion has begun, the rougher surfaces are made smooth by attrition, the friction decreases, and further motion occurs, gradually leading to the complete destruction of the structure.

54. The foregoing considerations will show the importance of taking all necessary measures to prevent settlement or crushing of the toe. Where the resultant pressure passes through the center of the base, the settlement is uniform, and does not cause serious harm; the nearer the resultant pressure is to the toe, the more uneven the settlement is likely to be; hence the importance, especially in soft soils, of making the eccentricity of the pressure as small as possible, or of providing for contingencies and uncertain conditions by the use of a large factor of safety.

CONSTRUCTION

MATERIALS USED

55. All classes of masonry materials—stone, brick, and plain and reinforced concrete—are used for the construction of retaining walls; and all grades are used according to the importance of the structure. In general, for important structures, a very good quality is employed and is really most economical, for the saving of quantity nearly offsets the difference in cost, leaving very little expense properly chargeable to the improved appearance and greater durability. Timber and steel bulkheads may also be classed with retaining walls, although their stability does not generally depend on their weight alone.

STONE AND BRICK

56. Stone.—Though dry rubble is sometimes used for retaining walls, and may show very good durability, the more common specifications call for either cut stone or a good quality of rubble laid in mortar; and, as will be seen by referring to Art. 45 and Table III, a very noticeable saving of masonry is considered a proper allowance for the use of the better quality. Referring to such better class of masonry, a few of the generally accepted requisites will be stated, allowing sufficient range to cover all that class for which the rule in Art. 45 requires only .35 of the height for the thickness of a vertical wall.

All stone should be sound and durable, free from seams, dries, shakes, and flaws, of large size, and laid on its natural bed in Portland-cement mortar, in horizontal courses not less than 12 inches thick. According to the appearance desired, these courses may be specified to be regular or broken. Breaks in courses should not be allowed more frequently than one break to 50 square feet of face. The whole wall should be well bonded together; not less than one-fifth of the face of the wall should consist of headers extending either entirely through the wall or to a depth of at least 4 feet. Headers should be evenly distributed throughout the structure, and rest on, and be covered by, stretchers so as to thoroughly bond the whole together. No stone (except raisers used to break courses) should have a less width than height, or a length less than two and one-half times its height.

All joints on the face of the wall should break not less than 9 inches, and all stone should be dressed to lay for the full bed, and for not less than 9 inches in from the face in the vertical joints with $\frac{1}{2}$-inch to 1-inch joints. Sometimes, joints $\frac{1}{4}$ inch wide are specified for retaining walls, but they greatly increase the cost, while actually reducing the strength, as the stone has to be dressed, and does not have so strong a grip on the mortar as does the rough stone.

57. The backing should be in courses of the same thickness as the face, and, with respect to headers and size of stone, should be as specified for the face. Less care, however, may be taken with the vertical joints, and the back may be left rough, except to within 4 feet of the top of the wall, which should be dressed smooth. All stone in the heart of the wall should conform in size to the dimensions specified for the face, except as modified by necessity, because less room is left between the face stone and the back stones. No space wider than 6 inches should be left to be filled with spalls, which should be laid in mortar; no spalls should be allowed in the bed joints.

58. All joints in the face should be dressed to a line and in the specified plane, and the rock face between joints should nowhere project more than 4 inches beyond the neat lines. The foundation below 1 foot under the ground surface may be built of stone of the same quality as specified for the backing, or even of a little poorer quality for the back portion. The coping course should be of selected stone, running the full width of the top of the wall, and projecting on the face from 2 to 6 inches beyond the neat line, and of a length suitable for stretchers; the top should be dressed to give no depression greater than $\frac{1}{2}$ inch below a true plane resting on the top.

59. Brick.—Brick may be used for small retaining walls, but is an unsatisfactory material for large walls. A greater thickness will be required for a brick wall than for one of a good quality of stone, or of concrete. A brick wall exposed to the weather should be capped with stone.

CONCRETE

60. Concrete, either plain or reinforced, is an excellent material for the construction of retaining walls, and one whose popularity is deservedly growing. No inferior material should be used for concrete retaining walls, and great care should be taken to get a compact body with a smooth

face; a wet concrete is to be preferred, as it is less likely to contain voids, and will also yield a better surface. The surface may be improved by greasing or soaping the inside of the form, and should be made richer than is necessary for the interior of the mass. This may be accomplished in either of two ways, both of which give very satisfactory results if well done, but are very unsatisfactory if poorly done.

61. In the first method, strips of sheet iron are provided, of a width a little greater than the thickness of the layers of concrete; before each layer of concrete is laid, the strips are placed 2 or 3 inches from the inside of the form, and this space is filled with the rich mortar, after which the layer of concrete is deposited to fill nearly to the top of the sheet-iron strips. The sheet is then raised nearly clear of the mass, and the concrete tamped to form a bond between the concrete and the mortar facing. The same operations are repeated for each course.

Fig. 19

With proper care, this method gives very satisfactory results, but it is tedious and expensive, and generally the necessary care can be secured only by constant watching. The strips may be made with handles attached, or with holes through which hooks may be inserted, or the upper edge may be bent to the form indicated in Fig. 19, to aid in drawing them up. The sections should not be made so long that they will be difficult to handle.

62. A much less expensive, and fairly satisfactory, substitute for the foregoing is by "spading a face." In this method, the concrete is first laid full against the form; then the flat blade of a spade is run down the inside of the form and the handle worked back and forth so as to press the concrete away from the form. The cement, being lighter than the sand and stone, will flow back more readily, and, separating from them, will go to enrich the surface. The loss to the mass is but slight, while the gain to the surface is important. In order that this method may be satisfactory, the concrete must be wet enough for the mortar to flow back against the form after the spade is removed. The

movement of the spade must be such that at different instants the concrete will be thrown back for the full height of the course; to do this properly, the blade should be placed flat against the form, as shown in Fig. 20 (*a*), and rocked back and forth, as shown at (*b*) and (*c*), so as to crowd in the concrete both at the top and at the bottom of the blade. Moreover, to produce satisfactory results at the middle of the course, the spade should be inserted to a less depth than the full length of the blade, as well as for its full depth.

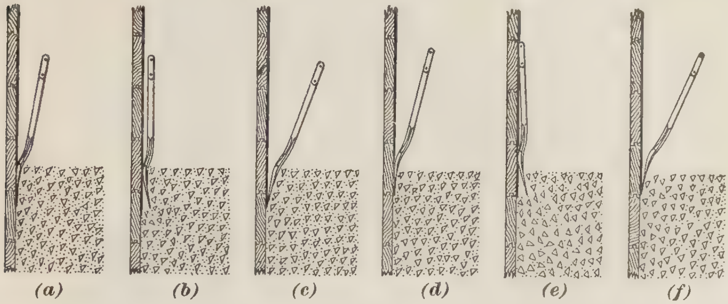


FIG. 20

A clearer idea of the movements may be obtained by reference to Fig. 20. A little practice will enable a workman to become very proficient, both as to speed and satisfactory results, working at three or four depths of penetration.

As this method is so much less expensive, and interferes so little with the rapid laying of the concrete, much less difficulty will be experienced in getting the work done according to specifications if spading is called for than if the separate layers of mortar were demanded. A face more like stone may be obtained by roughening the surface with a bush hammer or a similar tool.

63. Large stones may be thrown in with the concrete to advantage, both in economy and strength, when suitable stone is easily available. Some care should be taken when such stones are embedded, to avoid their coming in contact or forming large cleavage surfaces, for generally cement does not adhere to the surface of the stone as strongly as it coheres to other particles of cement (with some limestones

the case is the reverse). Concrete with large stones so embedded is called **rubble concrete** or **concrete rubble,** according to whether the concrete or the large stones predominate.

64. Reinforced Concrete.—By the use of reinforced concrete, a large saving of material may be accomplished; for, with the encased steel to take tension, thus efficiently tying the different parts together, a hollow shell may be used, and the weight of the backing may be utilized to oppose its own overturning pressure. The saving in cost by the use of the reinforcement will be much less than the decrease in volume, but will still be considerable in a wall of large size.

Fig. 21 shows a retaining wall of reinforced concrete. In the figure, (*a*) is the elevation of the face of the wall

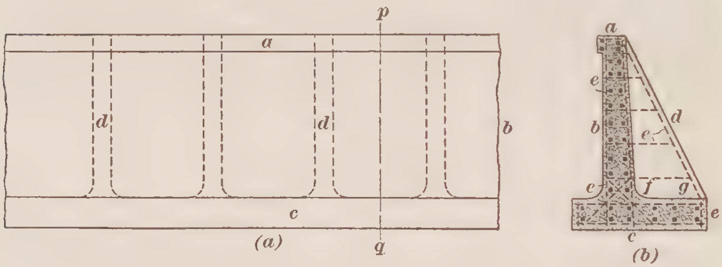


Fig. 21

and (*b*) is a cross-section on *pq*; *a* is the top, *b* the face, and *c* the base of the wall. The base is much wider than the thickness of the face; they are connected at frequent intervals by counterforts *d* that slope from the top of the wall to the inside line of the base. The face, base, and counterforts are reinforced with steel rods *e* to resist any tension and to prevent surface cracks. This is a very economical type of retaining wall, as the weight of earth that rests on the inner or back part *fg* of the base helps to maintain the stability; less weight of masonry is therefore required.

65. Forms.—The forms for concrete retaining walls are generally of such size and extend to such height above

the ground surface that it is not practicable to brace them efficiently from without, so that ties through the wall become necessary. Plain rods with screw ends and nuts are frequently used, and the ends are cut off a little within the plane of the surface of the concrete after the forms are removed. Some engineers put the rods through pipes that are 2 or 3 inches shorter than the thickness of the concrete; when the form is removed, the rod may be drawn out and

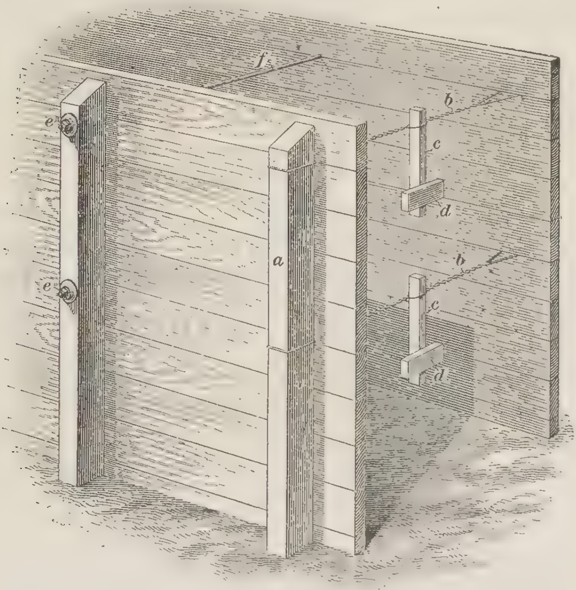


FIG. 22

the pipe left in place, by simply breaking the concrete loose from the rod where not covered by the pipe. Telegraph or similar wire is also used, the ends being tied together and then drawn taut by inserting plugs and twisting; several strands of wire may be used if required, and the wires may be cut off just inside the planes or faces of the wall. After cutting the rods or wires, or withdrawing the rods from within the pipes, the holes left in the face of the concrete may be plastered up. Fig. 22 illustrates both rod and wire

ties; the rods are shown at *f*, the ends being held by nuts and washers *e*, *e*; the wires are shown at *b*, *b*, the ends being looped around the posts *a*. The sticks *c* and *d* are for the purpose of twisting the wire so as to bring the sides to the proper distance apart and hold them there.

66. Before the concrete is placed in the forms, braces may be used on the inside to hold the sides out to their proper place, which is as important as the duty of the rods or wires to prevent their spreading too much. As the work progresses, these braces may be removed and the concrete may be relied on for this duty.

FOUNDATION

67. A good foundation for a retaining wall is as important as that the wall itself should be properly designed and built. *Nearly all the failures of retaining walls are due to poor foundations and improper design of the base of the wall.* Before the wall is built, a thorough examination of the character of the soil should be made, as explained in *Foundations*, Part 1. It should be borne in mind that sand, especially quicksand, is, as a rule, a very treacherous and unreliable material. Both sand and gravel, however, are excellent mixing materials for increasing the bearing power of soft clays and loam.

Generally, foundations have to be carried to a considerable depth below the ground surface, so as to avoid the dangers of frost and scour. In the northern United States, 4 feet below the surface is a common minimum requirement. Often, this is also of assistance in maintaining the stability of a retaining wall against sliding, as the coefficient of friction of the masonry on many soils is much less than that of two courses of masonry on each other—especially if the courses are laid in mortar, which binds them together. Considerable assistance in resisting sliding is gained by a few feet depth of earth in front of the walls above the bottom of the masonry.

68. On account of the greater pressure at the toe of a retaining wall, it is customary and advisable to enlarge the footing course by offsetting at the front below the ground level, as shown in Fig. 23 (*a*). On soft soil, this offsetting may be made of considerable length by making the depth below the surface greater than would be otherwise required. The offsetting may be made by a succession of steps, as in Fig. 23 (*b*), each with a rise about equal to the offset. With strong stone or other good material, the spread may be at a greater rate than 1 to 1, while with poor material it should be less. Every inch of such offset at the front of the foundation is a valuable aid to the stability of the wall as to overturning, and might well be taken advantage of to a greater extent than is customary.

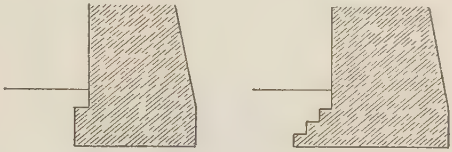

(*a*) (*b*)

Fig. 23

Walls are sometimes built with a similar offset at the heel; but this is of little use, simply adding a little to the weight and slightly to the leverage going to make up the moment of stability; while the same offset at the toe increases this moment several times as much.


(*a*) Fig. 24 (*b*)

69. To prevent sliding, the footing of a retaining wall is sometimes built on an incline, as indicated in Fig. 24 (*a*). Generally, this is of doubtful value, because of the compressibility of the soil. The inclined beds are objectionable, because of the facility with which moisture gathers in them. The design shown in Fig. 24 (*b*) is a better one.

70. As the toe of a retaining wall receives so great a load, piles are sometimes used to relieve the soil at this point (see Fig. 25). At best, this is an unsatisfactory arrangement, because by it, while the back edge of the wall rests on the soil near the natural surface, the support of the front is transferred to a lower level, and presumably a harder stratum of soil. If the upper soil is as soft as this expedient would suggest, there will be danger of settlement at the back, while the front is kept up to its original place,

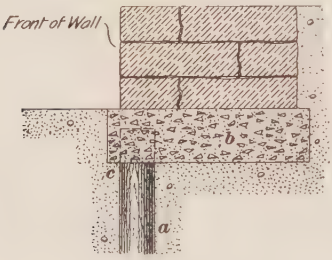


Fig. 25

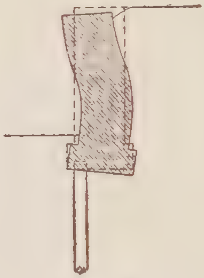


Fig. 26

and the result may be as indicated in Fig. 26, followed by the collapse of the wall. If the soil is sufficiently firm to prevent such settlement of the back, an offset at the toe can generally be more easily provided, and will be more efficient than piles. Piling should never be used except as a temporary expedient, unless the timber keeps constantly wet; otherwise, it will rot. When the piling keeps constantly moist, it may be used under the full width of the wall to give a foundation in soft soil, in which case it is well to give greater support to the front than to the back of the wall.

OTHER DETAILS

71. Coping.—The top of a wall, being more exposed to injury from the weather and other destructive agencies, should be made of carefully selected stone or of a richer concrete than is required for the body of the wall. This upper course or part of the wall is called the **coping** or

cap, and is generally given a width of about 2 or 3 feet. The coping should, if practicable, project from 2 to 6 inches (usually 3 or 4 inches) beyond the body of the wall, so that the drip will clear the face of the wall. If of stone, the coping should be made of large stone, of the full width of the course, and with as few transverse joints as possible. The top is frequently built with a slope to favor drainage.

72. Batter of the Face.—It has been explained that, for resistance to overturning, economy of quantity favors the use of a heavy batter for the face of retaining walls, even to the extent of using a triangular cross-section of wall. There are, however, serious objections to this form of construction: the earth near the surface is exposed to disturbing influences (chiefly frost in cold climates), which are more destructive to the stability of structures resting on, or otherwise brought in contact with these top strata, than to those either above ground or at a lower depth; for this reason, as well as from the natural objection to an acute angle or thin section, the top of the wall is never reduced to such an extent.

Aside from the objection to the thin top, an excessive batter exposes the face of the wall to undue destructive influences. With a vertical or slightly battered face, the rain or drainage from above passing over the face will run off rapidly with no injury to the wall; but with the greater slope, there is a greater inclination for the water to soak into the joints and seams, and do greater or less injury according to the quality of the construction, and according to whether freezing follows. Seeds, also, lodge in the joints and the resulting vegetation frequently causes serious damage to the wall.

Another serious objection to a large batter that frequently exists is the reduction of width available for use at the top of the wall.

73. Back of Wall.—The back of a retaining wall may preferably be left rough, thus increasing the friction of the filling. As an exception to this, the top part of the back (down to the frost line) should be left smooth, and preferably

with a forward slope, so that the frost and other surface influences will not cause any upheaval.

74. Bond.—Stone retaining walls should always be thoroughly bonded together, both transversely and longitudinally, so that the wall may act as a unit in resisting the thrust of the backing, instead of as a number of smaller units. The front part should be made of large and sound stone, not only for the sake of appearance, but also because the front is most strongly affected by the weather.

75. Drainage.—Most soils have a smaller angle of repose, and consequently cause a greater pressure on the wall, when thoroughly dry than when slightly damp; but, if saturated with water, they become not only heavier, but semifluid, and cause a much greater pressure. If a large retaining wall in a wet place is carried down to a subfoundation on impervious material, the escape of the water in the backing may be cut off and such an increase of pressure may follow. Care must be taken to avoid this danger by proper drainage, which may be effected by openings, called **weep holes**, through the wall. Weep holes are also sometimes provided to keep the water out of joints and seams in the wall itself.

76. Sea Walls.—As the earth behind a sea wall is generally saturated, it is heavier than the same filling would be if dry, and also naturally assumes a flatter slope (perhaps even nearly level); its pressure on the wall is greater and requires a greater thickness of masonry than would be necessary for an ordinary retaining wall.

77. Buttresses.—Buttresses are comparatively costly, and are seldom justified in the construction of ordinary retaining walls. For a very high wall, however, if economy of construction is the all-important factor, they may be used to advantage. As the extra cost per unit of volume is chiefly in the forms when concrete is used, a less height of wall may favor the use of buttresses with this material than where stone must be cut for all the extra angles of the buttresses.

The difficulty of satisfactorily bonding a buttress to the main wall, and generally the unsightliness and waste of room, add to the reasons for their rare use. Buttresses are sometimes used to stiffen old walls that have shown signs of impending failure.

78. Relieving Arches.—Masonry may sometimes be saved in retaining walls by the use of **relieving arches** (see Fig. 27). In this design, a series of transverse arches carry the earth, while the face wall has very little or no load. and is of use chiefly to close the openings and give an appearance of an ordinary retaining wall. The structure as a whole must be able, with the aid of the earth lying inside the arches, to bear the pressure of the earth on the back; as the weight of the structure is less than if the arches were filled, the width of the base must be materially greater than would be required for an ordinary wall.

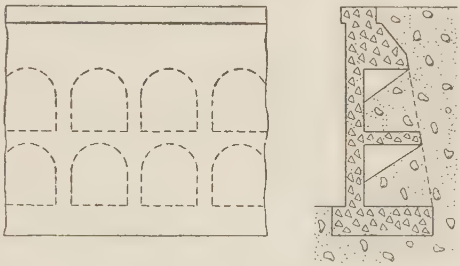


Fig. 27

Generally, the great expense of these arches far outweighs the saving of the material; but with the use of concrete, they may occasionally be advantageously employed for high walls.

CULVERTS

INTRODUCTION

1. Conditions to be Met.—Wherever the contour of the ground is such that the water even from a single storm will be dammed up by the embankment of a road, a source of danger to the bank is created, even if there is ordinarily no stream at the place. The magnitude of this danger varies with the nature of the material of which the embankment is made, with the amount and depth of the water, and especially with the relation of that depth to the height of the bank. If the water flows over the top of the embankment, the latter will quickly be washed away; and even at a considerably less depth, the material of the bank may become so softened by the water that it will have little strength to resist the pressure of the water, and its usefulness will be otherwise seriously impaired. In addition to the danger of destruction of the embankment, there may be damage done by flooding adjacent lands; and, in general, abutters are not slow to take advantage of the slightest occasion for a claim of damage against a railroad company.

To provide against these dangers, some way must be prepared for the passage of the water across the embankment. Where the possible volume of water is large, a bridge is usually necessary; where the volume is small, a *culvert* is sufficient.

2. Definition.—A **culvert** is essentially a small bridged passage for the conveyance of water across the line of a railroad or other way.

3. Location of Culvert.—The natural location of a culvert is usually at the lowest part of the embankment in which the water tends to collect. This is generally also the widest part of the embankment. Frequently, a culvert may be located at a slightly higher point, allowing the water to collect to a small depth before being carried away by the culvert, it being expected that the silt and débris brought down by the stream will soon raise the level of the ground to that of the bottom of the culvert. This will allow the use of a shorter culvert than would be required if the culvert were constructed at the lowest point; besides, a site may be selected that affords a better foundation.

4. Care must be exercised in the location of a culvert, lest the safety of the bank or other property be endangered, and lest opportunity be given the abutters for damage claims, either for flooding their land or for changing its surface, or possibly for the diversion of a natural stream. This danger of damage claims is not confined to the up-stream side of the bank, but is equally likely to exist on the lower side. Neither is the danger of such claims (which are upheld by the courts) necessarily governed by the actual damage done, for common law gives landowners a right to allow no interference with the processes of nature so far as it may affect their land. Actually, it might be a matter of indifference or even an advantage to an abutter to have the channel moved 50 feet to one side or the other, or to have soil deposited on or taken from his land; but if he objects to these alterations of natural conditions, his claim for damages will hold. The engineer should be careful not to divert a stream from the land of one owner to that of another, lest the one consider himself injured by receiving what the other claims to be injured by losing. For these reasons, it is seldom advisable to change materially the natural location of a culvert. The culvert, however, may be made to cross the line of the road at right angles, instead of on a skew, thus saving considerable expense.

Easy approach to the inlet of the culvert is necessary, especially where the volume of water is likely to be large or

the velocity great, in order that the channel may not become clogged or the water force a passage by some other way.

5. Culvert Avoided.—A culvert may sometimes be avoided altogether, or one culvert made to take the place of two, by excavating a ditch through a ridge separating two valleys. This plan is especially good where the general trend of the slope is parallel to the line of the road and is simply broken by a small spur or hill reaching just across the line.

6. Blind Drains.—Where the volume of water is very small, so that no danger to the bank or adjacent property need be feared, blind drains may be made by filling with loose rock, and allowing the water to percolate through the bank. These drains usually silt up, and soon become tight; but in the meantime the bank becomes more compact and less liable to injury.

CLASSIFICATION AND MATERIALS

TYPES OF CULVETS

7. Culverts are divided into four general classes; namely, *open*, *box*, *arch*, and *pipe* culverts.

8. Open culverts consist of abutments, with possibly one or more intervening piers, on which stringers or beams rest. These culverts, which are practically small bridges, are used where the depth of fill is insufficient to allow of a cover with filling over the top. The objections to them are the danger arising from their being open, and the interruption they cause in the smoothness of riding, on account of the fact that vehicles, in crossing them, pass from the ballast to a rigid support at the abutment, then to the elastic structure forming the bridge, then to another rigid support, and then to the ballast again. These successive changes are the more objectionable because they occur within a very short distance.

The abutments for open culverts are designed by the same rules as those for longer bridges, as explained in *Bridge*

Piers and Abutments. The stringers are designed as for small bridges.

9. Box culverts, Fig. 1, have side walls *W*, *W* supporting a horizontal covering *C* over which the fill is made. The bottom of the waterway *O* is usually paved. Box culverts are generally built of stone or wood, although concrete, usually reinforced with steel, is sometimes used.

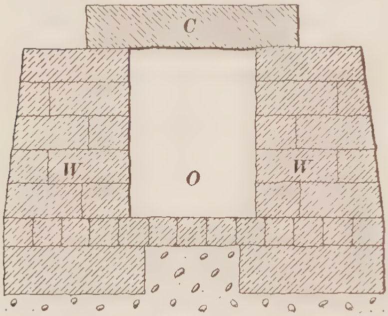


FIG. 1

10. Arch culverts are similar to box culverts, except that the horizontal roof is replaced by an arched cover, and the side walls are designed to resist the thrust from the arch. Arch culverts may be used for much larger openings than box culverts; in fact, the limit of practical construction with an arched roof is far beyond the limits within which the name culvert is generally used. At present, arch culverts are most frequently made of concrete, either plain or reinforced, although wood and rubble-stone masonry have been employed. Cut-stone masonry is also used for arch construction, but generally only for larger openings.

11. Pipe culverts are made by laying pipe either directly on the soil or in a bed of concrete. Pipes with as large a diameter as 5 feet have been used for this purpose. Pipe culverts are deservedly growing in popularity for small openings, because of the ease of construction, of the large amount of water they can convey, and of their small cost. They are generally made of cast iron or vitrified terra cotta.

12. Multiple Culverts.—The greater number of culverts have only one opening, and are known as **single culverts.** Frequently, however, a larger waterway is

required than would be practicable for a single opening, without changing the character of the structure; in that case, several walls are built to support the roof, and the space between each two walls forms an opening for the culvert. Such culverts are called **multiple culverts**, the special names **double** and **triple culverts** applying, respectively, to two and three openings.

MATERIALS

13. Terra Cotta.—Vitrified terra cotta is extensively used for pipe culverts, but is easily broken either by a blow or by a steady load. The breakage of pipe in transportation and handling is an important factor in the cost of culverts of this material. Being made in short lengths, the pipe yields readily to uneven settlement when laid on a soft foundation, thus reducing the breakage that might occur after laying if the lengths of pipe were as great as with cast-iron pipe. This uneven settlement, however, creates extra stresses of unknown magnitude. The settlement is also liable to open the joints or to cause water to collect in the low parts, where it may freeze and burst the pipe.

When terra-cotta pipe is used for culverts, it should be made thicker than would be necessary for sewer pipes, which are generally laid in trenches dug for the purpose, where little settlement is possible. Moreover, in sewer work the filling naturally arches over the top of the pipe, transferring much of its weight to the firm earth that has been undisturbed, and relieving the pipe accordingly. For culverts in a new fill, this relief is not to be expected.

14. Iron and Steel.—Cast iron is largely used for pipe culverts, and is superior to terra cotta, because it is stronger and less friable. With proper care in laying to prevent uneven settlement, cast iron makes a very durable culvert, provided that the depth of fill is sufficient to take up the impact from the trains. Wrought iron and steel have been used to some extent, but the corrosion of both these materials in the soil is rapid, and their use cannot be recommended

Considerable settlement may take place with an iron-pipe culvert without danger to the structure, provided that there are no abrupt changes in the support given to the pipe. Iron pipe in 12-foot lengths is very convenient for the construction of small culverts.

15. Timber.—In districts where stone is scarce and timber plentiful, wood is often used in making culverts, generally in the single- or double-box form. Timber culverts, however, are usually but a temporary expedient, and are frequently replaced by pipe or concrete after the road is opened for traffic and the timber has begun to decay. A timber culvert may last from 4 to 12 years before it needs renewing, the period depending on the character of the soil and climate, as well as on the kind and condition of the timber used. To allow for the decay of the timber, 2 or 3 inches extra thickness of wood should be provided. Timber has been used to some extent for arch culverts, but the conditions favoring this construction are infrequent.

16. Stone Masonry.—Where good stone can be easily and economically obtained, it is the favorite material for culvert construction. Unless the span is too long, the use of this material naturally suggests the box form, either single or double. Generally, a very low grade of material and workmanship has been used for this construction, and often the stones are laid dry (without mortar). Although there is no reason for the use of first-class stone, dressed with the care desirable for bridge piers and abutments, the use of poor stone is likely to prove a false economy, for a large expense is involved in repairing a break in a culvert, especially if the bank is high. Stone with seams should be particularly avoided for the cover, on account of the transverse stresses to which the roof is subjected. No stone whatever should be used that is likely to disintegrate in water. A great many failures of stone box culverts have been due to disintegration, or to the use of seamy cover stones. If stratified stone, however faint the stratification may be, is used, it should be laid on its natural bed, with the

strata horizontal, whether the stone is placed in the walls or in the cover.

17. With the present low prices of Portland cement, there is no real economy either in laying stone culverts dry or in using natural cement. Mortar composed of good material renders the structure, to some extent, monolithic, and therefore stronger; the whole structure thus becomes a unit instead of being made up of numerous units, each subject to settlement or stress without help from the adjacent units.

Although there are some natural cements that show good qualities, and under certain conditions may be as satisfactory as some brands of Portland cement, natural cement, being less uniform, is more uncertain, and a richer mortar should be used with it than with Portland cement. A 1 : 2 natural-cement mortar is commonly used as equivalent to a 1 : 3 Portland cement. Natural cement should never be used in freezing weather; good brands of Portland cement may be used, with proper precautions, in weather considerably below the freezing point. In such temperatures, or when such temperatures are expected before the mortar can set, salt should be added to the water with which the mortar is mixed. A rule that experience has proved satisfactory is to use 1 pound of salt to 18 gallons of water, for a temperature of 32° F., with an additional ounce of salt per 18 gallons for each degree below 32°. The proportion of salt should be based on the coldest weather expected between the time of mixing and the time of setting.

If the temperature is below freezing at the time of mixing, both the water and the sand should be warmed sufficiently to prevent all possibility of the freezing of the mixture before it is in place and past all danger of disturbance by the laying of stone on or near it. It is also important that the sand used should be clean, so that every grain of it will be entirely covered with cement.

CONCRETE

18. Plain Concrete.—Concrete, either plain or in combination with steel, is growing greatly in favor as a material for culvert construction, even frequently supplanting stone in regions where the natural conditions are especially favorable to the use of the latter material. Several causes have contributed to effect this change; namely, the low prices of cement and stone suitable for concrete, the ease and economy of transportation, the ease of handling and construction, and the fact that most of the work can be done by unskilled labor. To these advantages must be added the fact that concrete offers less resistance to the flow of water and catches less drift than a stone surface, thus materially increasing the capacity of the culvert for a given area.

What was said in Art. **17** regarding the selection of cement and sand for mortar, and the treatment of the ingredients in cold weather, applies with greater force to concrete culverts; for in stone masonry, if the cement is deficient, the structure will still have a greater or less stability, while with concrete the cement is the essential element, without which there would be nothing but a pile of stone and sand.

19. None but strong and hard stone should be used in making concrete for culverts, and it is preferable that the stone should be angular, because such form furnishes a better bond and better grip to the cement. "Crusher-run" (that is, stone broken to assorted sizes, just as it comes from the crusher, without screening) is more economical than stone of uniform size, and gives greater strength for the same nominal proportions, because the smaller stones partly fill the voids between the larger ones, leaving more cement to thoroughly coat the surfaces of the sand and stone, and make the bond more perfect.

Plain concrete lends itself readily to the arch form of culvert, for no cutting is required, as for stone. The cost is slightly increased by the price of forms; but, as they

may be used repeatedly, the extra cost they involve is of comparatively little moment.

20. Reinforced Concrete.—For culverts, as well as for other structures subjected to bending stresses, reinforced concrete is preferable to plain concrete. In reinforcing the concrete with steel, the latter is so placed that it will resist the tension only, and the concrete the compression only, to which the structure is subjected.

The use of reinforced concrete permits of larger culvert openings than would be economical with either plain concrete or stone; so that a single larger opening may frequently take the place of the double culvert that might be the economical design with the use of either of the latter two materials.

Concrete is strongly alkaline, and so serves as a protector from the corrosive elements that tend to destroy the steel. Though, of course, greater safety is insured where there are no cracks in the concrete, so that no air will reach the steel, it is very probable that, unless the width of a crack is large compared with the depth, the air so reaching the steel will be attacked by the concrete and lose its corrosiveness.

21. Several styles of reinforced-concrete culverts have been built, both with arched and with flat roofs. With small spans, however, and with the usual depth of fill above, the possibility of great tensile stresses developing in an arch is so remote that reinforcement is not necessary in concrete arch culverts. Small rods, wire netting, and expanded metal are sometimes used for the purpose of preventing cracks in the surface. Longitudinal reinforcement in the invert may be conducive to economy and strength where the foundation is soft.

In flat-top or box culverts of reinforced concrete, the reinforcement consists of old rails or I beams, or of rods, wire netting, or expanded metal. Steel rods or bars are either employed in their usual form, or are twisted, corrugated, or in some other way prepared so as to form shoulders in the metal to give the concrete a firmer grip than that offered by the simple adhesion between the concrete and the metal. When reinforced concrete is used, the walls and pavement

are also generally made with the same combination of materials. Plain-concrete walls are frequently combined with a reinforced-concrete cover.

22. Most of the methods of combining steel and concrete being patented, the engineer should make sure of his right to use the form of construction he wishes to adopt. Generally, this requires simply the purchase from the patentee of the form of steel to be used, the royalty for its use being included in the price of the steel. In some cases, a patent covers only the method of using simple materials to be purchased in the open market, and the right to employ that method must be obtained, by purchase or otherwise, from the patentee. Some patent claims conflict with one another, and the designer, in order to avoid trouble, should investigate the matter thoroughly.

GENERAL PRINCIPLES OF DESIGN AND CONSTRUCTION

SIZE OF WATERWAY

ELEMENTS DETERMINING THE SIZE OF WATERWAY

23. The size of the opening, or **waterway**, of a culvert is determined by the maximum amount of water that may pass through it in a given time, and also by the character of the culvert itself—its approaches and outlet.

The factors governing the volume of water that may pass through the culvert in a given time are: (1) *the drainage surface covered*, including: (*a*) its area, (*b*) its shape, and (*c*) its watercourses; (2) *the rate of rainfall:* (*a*) for a brief interval, and (*b*) for a longer period, also (*c*) the accumulation of snow; (3) *the character of the soil*, including: (*a*) the formation of the soil, and (*b*) the vegetation; (4) *the natural slope of:* (*a*) the bed of streams and rivulets contributing to the flow through the culvert, and (*b*) the surface of the land adjacent to these streams.

24. Drainage Surface.—With other factors unchanged, a larger culvert is required where a large than where a small drainage area is covered; but, for several reasons, the required waterway is by no means proportional to the area drained, even if other factors remain unchanged. In the first place, a larger rainfall per unit of area may be expected on a small surface than on a large one. In the second place, the conditions governing the speed of run-off being the same, it takes longer for water from the extremities of a large area to reach the culvert than from the extremities of a small area. Therefore, unless the storm is prolonged and remains of uniform intensity, the water from those parts of a large area that are near the culvert runs off before the water from the remote parts reaches the culvert. There will be less danger of the accumulation of water from a long and narrow drainage surface than from a surface of equal area but of more nearly square or circular form. In the third place, not all the rainfall is included in the surface discharge, and much that eventually reaches the culvert is retarded by being temporarily absorbed by the ground. This prolongs the time during which the storm water reaches the culvert, and reduces the amount to be discharged in a certain time. Finally, as water flows more freely when gathered in streams than when flowing over the surface of the ground or percolating through it, the water from a storm reaches the culvert sooner where the basin has many watercourses. The distribution of these watercourses in the basin should also be considered.

25. Rate of Rainfall.—In designing a culvert, provision must be made for the severest storms. In many localities, few data are available to indicate the maximum rainfall; and, where records have been kept, they frequently give only the daily precipitation, and usually cover comparatively few years. For a culvert draining a large area, the figures from these records may be a reasonable guide for determining the required waterway. It must be kept in mind, however. that the severity of a storm varies not only for different

places within a given area, but at different times during the storm. Of a storm lasting a full day or more, one-half the rain may fall in one hour, or even in a few minutes.

26. In cold latitudes, the accumulation of snow that may be melted by a warm rain is of much importance in its relation to culverts. As the spring rains are frequently the most severe, and also release the accumulations of the winter's snow, it is generally at that season that a culvert is most taxed, especially if the surface of the drainage area is bare and frozen.

27. Character of Soil and Vegetation.—The nature of the soil, and the quantity and character of the vegetation in it, have a considerable effect on both the velocity and the amount of water that reaches the culvert. Rocky or porous soils without vegetation allow a larger and more rapid flow than clay or loam soils covered with vegetable growth.

28. Slope of Ground and Streams.—If the courses of the brooks and rivulets are rapid and the surface of the adjacent ground is steep, the flow of a stream and the velocity of its discharge are greatly increased. If the more distant parts of the valley are precipitous and open, while those nearer the culvert are flat and covered with vegetation, the rain falling on all parts will reach the culvert more nearly at the same time than if the ground in the neighborhood of the culvert is steep and otherwise adapted to easy and rapid flow.

29. Character of the Culvert and Immediate Surroundings.—The required waterway depends to a great extent on the character of the culvert itself and of its immediate surroundings. A steep grade in the culvert increases the discharge. The roughness of the inner surface of the culvert is another very important factor: the smoother that surface, the less is the danger that the culvert may be clogged with drift, and the less the resistance of friction, as well as the resistance caused by eddies. The position and shape of the inlet are also of importance; they

should be such that the water will form no eddies on reaching the culvert. Of equal or greater importance is the outlet, for no more water can pass through a culvert than can run out of it.

30. If the water can be allowed to back up, so that it will flow through the culvert under a head, the volume that can be passed through a culvert of given size and character is greatly increased. It is not always possible, however, to allow this backing up of the water without danger of damage to adjacent property or to the embankment itself through which the culvert passes.

FORMULAS FOR SIZE OF WATERWAY

31. Introductory Remarks.—A theoretically perfect formula for determining the size and shape for the opening of a culvert would take account of all the factors mentioned in Art. **23**; but a knowledge of the effect of some of these factors is not yet sufficiently exact to make such formula possible.

It is not practically worth while to make an exhaustive study of all the conditions and records in order to reduce the size of opening to the minimum size that is consistent with safety. The conditions may be different in the future from those that have existed in the past, and the expense of an exhaustive study may exceed the extra cost of a more liberal design. It is seldom advisable to reduce the size to the very last inch given by formulas; for those formulas are but approximations founded on uncertain data and assumptions.

32. The formulas most commonly used for determining the size of culverts take account only of: (1) the area and form of the drainage surface; (2) the character of the soil, including vegetation; and (3) the slope of the ground. They are mainly empirical, and serve only as general guides.

Where sudden and severe storms are common or likely to occur, proper allowance must be made. In this, the engineer must use his judgment. Generally, very arid regions

are so flat and their soil is so porous that no culverts whatever need be provided.

33. Myer's Formula.—The formula most generally used for determining the waterway for a culvert was constructed by E. T. D. Myer, and is as follows:

$$A = c\sqrt{M}$$

in which A = area, in square feet, of required opening;
M = area of drainage basin, in acres;
c = a coefficient varying with slope of ground, character of soil, and amount and character of vegetation.

The following are the values of c commonly used: $c = 4$ for mountainous and rocky ground without vegetation; $c = 1.5$ for hilly ground covered with verdure; and $c = 1$ for slightly rolling prairie.

EXAMPLE.—What size of culvert with a rectangular opening is required for an outlet for a drainage basin 4,000 feet long and averaging 900 feet in width, the ground being hilly and under cultivation?

SOLUTION.—Substituting in the above formula 1.5 for c and $\frac{4{,}000 \times 900}{43{,}560}$ for M,

$$A = 1.5\sqrt{\frac{4{,}000 \times 900}{43{,}560}} = 13.65 \text{ sq. ft.}$$

A box culvert $3\frac{1}{2}$ ft. $\times$ 4 ft., whose area is 14 sq. ft., might be used, but generally there is no intermediate size for box culverts between 3 ft. $\times$ 4 ft. and 4 ft. $\times$ 4 ft., so that a culvert with a 4-ft. square opening would be used in this case. Ans.

34. Talbot's Formula.—A later, and generally more satisfactory, formula has been derived by Prof. A. N. Talbot, of the University of Illinois. The significance of the symbols A, M, and c are as in Myer's formula, but Professor Talbot gives the following values for c *when used in his formula* (note that the substitution of these values for c in Myer's formula would not be permissible): $c = \frac{2}{3}$ to 1 for steep and rocky ground; $c = \frac{1}{3}$ for rolling agricultural country subject to floods at times of melting snow, and with the length of valley three or four times its width; $c = \frac{1}{5}$ to $\frac{1}{6}$ for

districts not affected by accumulated snow, and where the length of the valley is several times the width.

Talbot's formula is as follows:

$$A = c\sqrt[4]{M^3}$$

EXAMPLE.—What size of culvert would be required, according to Talbot's formula, for the same conditions as in the example in Art. **33**, with probabilities of floods at times of melting snow, and valley of moderate length?

SOLUTION.—These conditions suggest a value of c, for use in this formula, of $\frac{1}{2}$. Substituting known values in the formula,

$$A = \frac{1}{2}\sqrt[4]{\left(\frac{4{,}000 \times 900}{43{,}560}\right)^3} = 13.7 \text{ sq. ft.}$$

This agrees closely with the result by the use of Myer's formula, and a 4-ft. square opening may be used. Ans.

35. Comparison of Formulas.—Table I gives areas of waterway as computed both by Myer's and by Talbot's formulas:

TABLE I

Drainage Area Acres	Steep and Rocky Square Feet		Rolling Prairie Square Feet	
	Myer	Talbot	Myer	Talbot
1	4.00	.67 to 1.00	1.00	.17 to .33
10	12.65	3.75 to 5.62	3.16	.94 to 1.87
100	40.00	21.10 to 31.60	10.00	5.27 to 10.54
1,000	126.00	119.00 to 178.00	31.60	29.60 to 59.30
10,000	400.00	667.00 to 1,000.50	100.00	167.00 to 333.00

It will be noticed that Myer's formula gives larger openings when the drainage area is less than from $\frac{1}{6}$ to 2 square miles (according to whether the smaller or the larger coefficient is used with Talbot's formula), while for greater basins Talbot's formula gives larger results. In applying Myer's formula for large basins, considerable liberality should be used, especially if the upper part of the valley has steeper slopes than the part nearer the culvert.

For a general formula, Professor Talbot's seems more satisfactory and suited to a larger range of conditions than Myer's, but with small basins it would seem wise to provide a considerably larger opening than the formula calls for. This increase in size would be adopted as much because the increased cost is slight, compared with the increased safety, as because the figures seem unreasonably small. It should be borne in mind that both Myer's and Talbot's formulas are but rough approximations, and are intended, as Professor Talbot himself says, "more as a guide to the judgment than as working rules."

PRACTICAL METHOD

36. The waterway of most culverts is determined by the so-called "practical method," and not by any formula. This method consists in a study, made on the ground itself, of all the conditions, and a comparison with culverts built under similar conditions. An experienced engineer can in this manner estimate very closely the duty that the culvert is to perform, and decide on its dimensions accordingly.

In studying the ground, note should be taken of the past high-water marks, of the cross-section that the stream must have had at such times, and also of whether the current seems to have been rapid at the time of high water. Signs of erosion and drift are helps in this study of the ground; existing openings, if any, should also be noted, and the "oldest inhabitant" should be interviewed. All these signs (except the last) are liable to be underestimated; so a liberal allowance should be added to what they would seem to indicate. If the line is through new country, it should be kept in mind that, as the region is built up and forest and other vegetation is decreased, a larger and more rapid run-off is to be expected.

In designing culverts, it is well to remember that one washout of the bank will cost much more than the increased cost of a larger culvert, even disregarding the delay to traffic that would result, and the possible loss of human life.

For practical reasons, no covered culvert should be built less than 2 feet wide by 3 feet high, because of the danger of clogging up and the difficulty of effecting the consequent clearing out.

COVERS OF BOX CULVERTS

37. Formula for Thickness.—The thickness of cover required for a box culvert is governed by: (1) the amount and character of the load it must bear; (2) the span, or width of opening; (3) the transverse strength of the material of which the cover is composed.

The required thickness may be determined by the formula

$$t = \sqrt{\frac{3\,W l^2}{4\,j_1 R}}$$

in which t = thickness of cover, in inches;

W = weight, in pounds per square foot, of uniformly distributed load (or the equivalent distributed load if the loads are concentrated);

l = effective span, in feet, which may ordinarily be taken as $1\frac{1}{4}$ times the clear opening;

R = modulus of rupture of the material used, in pounds per square inch;

j_1 = reciprocal of factor of safety ($j_1 R$ = safe intensity of stress in bending, in pounds per square inch).

38. Loads.—The load to be carried is the weight of the bank above the cover and the heaviest load (usually a locomotive) that is likely to be on the bank, with the proper allowance for the effect of impact and imperfect distribution of the load. The weight of materials of which such banks are usually composed varies from 75 pounds per cubic foot for dry loam to 140 pounds per cubic foot for quartz sand thoroughly wet, probably seldom reaching either of these extremes for the average of the whole height of embankment; it may generally be taken with safety as 125 pounds per cubic foot.

The weight on the drivers of the heaviest locomotive in common use may be taken as 50,000 pounds per axle, with axles 5 feet on center, or an average of 10,000 pounds per foot of track. Though the rails and ties do not make a perfect distribution of this load, a few feet depth of fill will probably distribute the load over a width of at least 10 feet, making 1,000 pounds per square foot, and absorb the effect of impact. This load may be safely used for the design of culvert covers if the height of bank is 10 feet or more. With a lower bank, the distribution of the live load will be less perfect, and the effect of impact will be greater; so that it will be advisable to design the cover as if the actual height of the embankment were 10 feet, with the distribution of the live load as before. This will give for W, in Art. **37**, a value of 2,250 pounds per square foot, being the sum of the live load (1,000 pounds per square foot) and the weight (125×10) of the assumed embankment. It should be understood that this height of embankment is assumed merely for the purposes of calculation.

Though it is not explicitly acknowledged by engineers, it is the generally accepted opinion, which accords with common practice in dimensioning, that the arching over of the materials of the bank effects a considerable reduction in the stress developed in the cover, and that a cover calculated for the full load of a 10-foot bank and a live load as just given will be ample for any depth of fill, unless the bank is composed of sand or soft clay. Sand, when either dry or thoroughly wet, has very little tendency to arch over; when moderately damp, it will do so. Accordingly, a load of 2,250 pounds per square foot may be used, except when the bank is composed of sand or soft clay.

39. Working Stresses.—The modulus of rupture of different stones used for culvert work varies greatly, even for apparently perfect and uniform stone of the same class. The variation, in pounds per square inch, is from 900 to 2,700 for granite, with an average of 1,800; 550 to 2,400 for sandstone, with an average of 1,250; 200 to 2,500 for limestone,

with an average of 1,500; and 400 to 4,500 for bluestone flagging, with an average of 2,700.

It is not safe to use so large a modulus of rupture as the average, unless a larger factor of safety is also used. With proper care in culling out all stone showing defects, it is safe to use, for working stresses ($= j_1 R$, or the modulus of rupture divided by the factor of safety), 450 pounds per square inch for granite or bluestone flagging, and 250 pounds per square inch for sandstone or limestone of good quality. By substituting the values given, namely:

$W = 2{,}250$ (unless the fill is of sand or soft clay, in which case 125 pounds per square foot should be added for each foot of depth of fill over 10 feet);

$$j_1 R = \begin{cases} 450 \text{ for granite or bluestone,} \\ 250 \text{ for good quality of sandstone or limestone;} \end{cases}$$

we have, for granite or bluestone,

$$t = \sqrt{\frac{3 \times 2{,}250\, l^2}{4 \times 450}} = 1.94\, l \qquad (1)$$

and for sandstone or limestone,

$$t = \sqrt{\frac{3 \times 2{,}250\, l^2}{4 \times 250}} = 2.60\, l \qquad (2)$$

40. It is customary to make the thickness of the cover stone for box culverts equal to one-fourth the width of opening (or 3 inches per foot) for granite, and one-third (or 4 inches per foot) for sandstone and limestone; this makes an allowance for the spaces between cover stones. Bluestone makes a very good cover for small openings, but can seldom be obtained in good condition large and thick enough for large culverts.

41. Materials for Covers.—Cover stones should have a good level bearing on each wall of not less than their depth, with a minimum of 12 inches. Covers are often laid with open joints between the stones, it being claimed that this facilitates the drainage of the fill above. This construction, however, is objectionable, because it tends to cause clogging of the culvert.

42. Hard-pine timber used for culvert covers is subject to rapid decay, and at least 3 inches extra thickness should be allowed for it, with an intensity of stress of 800 pounds per square inch for the remainder of the timber.

43. Plain concrete is not generally used for the coverings of box culverts, because, for a given size of opening, an arch is stronger and not more expensive. Plain concrete may, however, be used with working stresses of 150 pounds per square inch for a 1 : 2 : 4 concrete, or 75 pounds per square inch for a 1 : 3 : 6 concrete (a good brand of Portland cement with proper sand and broken stone or gravel being used).

For good concrete reinforced with rods of mild steel, well designed and properly mixed and placed, the working stresses, in pounds per square inch, may be 12,500 to 15,000 pounds tension on steel rods; 625 pounds compression on concrete for a 1 : 2 : 4 mixture; and 500 pounds compression on concrete for a 1 : 3 : 6 mixture.

SIDE WALLS OF BOX CULVERTS

44. The thickness of the side walls of box culverts should, theoretically, be determined by the same rules that are applicable to retaining walls, but the effect of the bank over the top of the culvert must be kept in mind. The resistance to the horizontal thrust from the surcharging bank, given by that portion of the bank which is on the other side of the culvert and lies above the cover, greatly reduces the thickness of the wall required from what would be needed at the bottom if the wall were carried up to the top of the fill. Experience indicates that the thickness of the side walls of culverts, with nothing to serve as bracing to hold them apart either at top or bottom, should not be less at the top than one-half the height of the culvert, nor at the bottom less than two-thirds that height.

In timber culverts, the walls not being designed to resist by their weight the thrust of the bank, as must be done with stone culverts, it is very important that the walls should be securely drift-bolted together, as well as to the pavement (or sills) and the roof. With walls properly so drift-bolted,

the side walls may be made of a uniform thickness of one-fourth the height of the culvert, with 2 or 3 inches additional for decay.

45. In stone culverts, all the stone in the side walls should be sound, roughly squared, and laid so as to be well bonded together. Not less than one-half of the top course, and one-quarter of the lower courses, should pass entirely through the walls; these through stones should be of good size and be selected especially for the duty of binding the wall together.

In double or multiple culverts, the thickness of the partition wall should not be less than one-half the height, and must provide ample bearing for the ends of cover stones. All the top course and not less than one-half the bulk of the lower courses of these partition walls should be of through stones; generally, all courses may be through stones.

DIMENSIONS OF ARCH CULVERTS

46. Definitions.—The parts of an arch that are now to be defined are illustrated in Fig. 2.

The **span** $a b'$ is the perpendicular distance between the faces of the walls A and B supporting the arch.

47. The **springs**, or **spring lines**, are the lines of junction of the arch with the supporting walls; these lines run longitudinally with the arch, and are represented in the cross-section by the points a and b'.

48. The **crown** $d e$ is the central and highest part of the arch.

49. The **rise** $c d$ is the vertical height from the level of the spring lines to the lower face of the crown; it is the middle ordinate of the arc $a d b'$ forming the under face of the arch.

50. The **soffit** is the lower concave surface of the arch, and is represented in the figure by the line $a d b'$. The soffit is also called the **intrados**; the latter term is often applied to the line of intersection, as $a d b'$, of the soffit with a vertical plane normal to the axis of the arch.

51. The **extrados** is the outer convex surface of the arch, or the line of intersection of this surface with a vertical plane normal to the axis of the arch (the line *s e t′*, or the surface of which this line is the projection). When the arch stones, or **voussoirs**, are distinct from the spandrel filling, as is usual with stone arches, the joint *u e v* between the arch stones and the spandrels is more properly called the extrados. Even though the structure may be monolithic, the same imaginary surface or line is also so designated for mathematical purposes. The term **back** is also applied to the extrados surface.

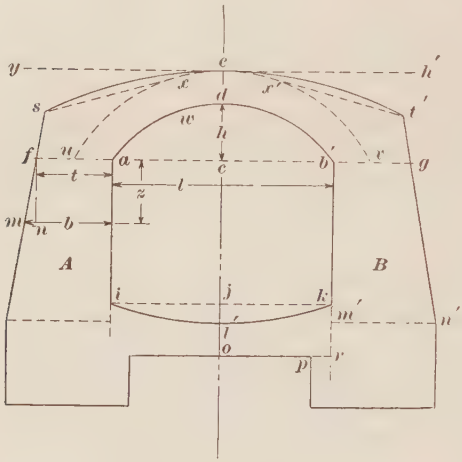


FIG. 2

52. The **haunches** are the parts of the arch lying between the crown and the springs. There is no definite division line between the crown and the haunch; the latter term may properly be applied to the part *a f s x w*, or may be used for the whole area *a f s e d* to the center of the arch.

53. The **key** is the central highest part of the arch. When the arch is composed of separate stones, the stone at the key is called the **keystone.**

54. The term **spandrel** is applied to the space over the haunches of the arch and below the level of the top of the crown. In Fig. 2, *u e y* and *v e h′* are the spandrels. The material occupying this space, whether of masonry or earth, is called the **spandrel filling.** The parts *u e s f* and *v e t′ g* of the spandrel filling that are composed of masonry are called **backing.**

55. The **invert** is the *inverted arch* $i\,l'\,k$ at the bottom of the culvert, and is subjected to stresses similar to those in the main arch, but in the opposite direction. The difference $j\,l'$ in level between the spring lines and the crown of the invert is called the **dip** or **dish** of the invert, and corresponds to the rise of the upper arch.

56. Radius of Intrados.—Arches for culverts are usually made in the form of a circular arc. If the arc is semicircular, the arch is called a **semicircular** arch; if the arc is less than a semicircle, the arch is a **segmental** arch. The semicircular form is more common, although, theoretically, the segmental form is the better form.

The following formula gives the radius R of the intrados $a\,d\,b'$, Fig. 2:

$$R = \frac{h}{2} + \frac{l^2}{8\,h} \qquad (1)$$

in which h = rise $c\,d$;

l = span $a\,b'$

If the arch is semicircular,

$$R = h = \frac{l}{2} \qquad (2)$$

57. Depth of Key.—The factors governing the thickness of an arch at the key and in the haunches are: (1) the amount and character of the load the arch must bear, which is the same as though the culvert were of box form (see Art. **38**); (2) the span; (3) the rise; and (4) the compressive strength of the material of which the arch is made. These are the factors governing the thickness of an arch; but in the common formula for determining the depth of key, the load and the strength of material do not directly appear; however, they were taken into account in selecting the constants contained in the formula. This formula, which is mainly empirical, was first suggested by John C. Trautwine in his well-known Pocketbook, and is now very extensively used; it is as follows:

$$K = \frac{\sqrt{R + \frac{l}{2}}}{4} + .2 \qquad (1)$$

Here R and l have the same significance as in Art. **56**, both being in feet; and K is the depth of key, in feet. If the arch is semicircular, $R = \frac{l}{2}$, and the formula becomes

$$K = \frac{\sqrt{l}}{4} + .2 \qquad \textbf{(2)}$$

These formulas apply to first-class cut-stone arches. For second-class stonework, the results should be multiplied by $\frac{9}{8}$, and for rubble, brick, or concrete, by $\frac{4}{3}$.

EXAMPLE 1.—The span of an arch is 8 feet, and the rise, 1 foot 9 inches; to determine the radius.

SOLUTION.—Here, $h = 1.75$ and $l = 8$. Substituting these values in formula **1**, Art. **56**,

$$R = \frac{1.75}{2} + \frac{8^2}{8 \times 1.75} = 5.45 \text{ ft. Ans.}$$

EXAMPLE 2.—The span of an arch is 6 feet, and the rise, 1 foot 6 inches; what depth of key should be used for second-class stonework?

SOLUTION.—To apply formula **1**, Art. **57**, we have $l = 6$ and $R = \frac{1.5}{2} + \frac{6^2}{8 \times 1.5} = 3.75$. Substituting in the formula and multiplying by $\frac{9}{8}$ for second-class work,

$$K = \frac{9}{8}\left(\frac{\sqrt{3.75 + \frac{6}{2}}}{4} + .2\right) = .96 \text{ ft., or } 11\tfrac{1}{2} \text{ in. Ans.}$$

58. Thickness of Walls.—The thickness of the abutments, or side walls, of arch culverts required to resist the thrust of the arch may be found by the following formulas (see Fig. 2):

$$t = \frac{R}{5} + \frac{h}{10} + 2 \qquad \textbf{(1)}$$

$$b = t + \frac{l\,z}{24\,h} \qquad \textbf{(2)}$$

in which R, h, and l have the same significance as in Art. **56**; t represents the thickness, in feet, of the abutment at the level of the spring lines; and b is the thickness, in feet, of the abutment at a distance of z feet below the spring line.

These formulas give the thickness of abutment necessary to resist the thrust of the arch. But if the thickness so

obtained for the bottom of the abutment is less than two-thirds the height of the wall to the spring line, it will be insufficient to resist the thrust of the bank, and the thickness at the bottom should be increased to two-thirds of the height, while the top should be thickened by the same amount.

The required thickness of abutments is independent of the quality of masonry, except that, if voids exist, allowance must be made for them.

If the arch is semicircular, $R = h = \frac{l}{2}$, and formulas **1** and **2** become, respectively,

$$t = \frac{3\,l}{20} + 2 \qquad (3)$$

$$b = t + \frac{z}{12} \qquad (4)$$

59. Batter.—The inner surface of each wall is made vertical; the back of the wall, inclined (see Fig. 2). The batter of the back is expressed as follows: In the vertical distance $f\,n$, or z, the back deviates from the vertical by the amount $m\,n$, or $b - t$; therefore, the horizontal deviation I, in feet per foot vertical, is given by the equation

$$I = \frac{b - t}{z}$$

which, by inserting the value of b from formula **2** of Art. **58**, becomes

$$I = \frac{l}{24\,h} \qquad (1)$$

For the batter i, in inches horizontal per foot vertical, we have

$$i = 12\,I = \frac{l}{2\,h} \qquad (2)$$

Example 1.—What thickness at top and bottom is required for the wall of an arch of 6 feet span and 1 foot 6 inches rise, the walls being 4 feet 6 inches high to the spring line?

Solution.—Here, R (as found in example 2, Art. **57**) = 3.75 $h = 1.5$, $l = 6$, and $z = 4.5$. Then, using formulas **1** and **2**, Art. **58**,

$$t = \frac{3.75}{5} + \frac{1.5}{10} + 2 = 2.9 \text{ ft.}$$

and $$b = 2.9 + \frac{4.5 \times 6}{24 \times 1.5} = 3.65 \text{ ft. Ans.}$$

EXAMPLE 2.—What batter should be given to the back of abutments for arches of which the ratios of span to rise are: (*a*) 2? (*b*) 4? (*c*) 8?

SOLUTION.—(*a*) Since the ratio of the span to the rise is 2, $\frac{l}{h} = 2$. Substituting in formula **2**, $i = \frac{2}{2} = 1$ in. horizontal to 1 ft. vertical. Ans.

(*b*) In this case, $\frac{l}{h} = 4$. Substituting in formula **2**, $i = \frac{4}{2} = 2$ in. horizontal to 1 ft. vertical. Ans.

(*c*) Similarly, $i = \frac{8}{2} = 4$ in. horizontal to 1 ft. vertical. Ans.

60. Thickness of the Haunches.—The thickness of the wall at the level of the spring line having been determined, as well as the batter of the back, this batter should be continued upwards to a point *s*, Fig. 2, midway between the level of the springs and that of the extrados at the crown. From this point, the top of the haunches may be made a plain surface tangent to an extrados *u e v* made concentric with the intrados from the crown to its intersection *x* with the said plain surface, the top taking the form indicated by the dotted line $s\,x\,e\,x'\,t'$. It is customary (especially if concrete is used) for such small arches as are used in culverts to make the top a cylindrical surface, as indicated by the full line $s\,e\,t'$; this takes but little extra material with so small an arch, and makes a better and more easily defined surface.

If the structure is to be of stone masonry, the part outside the line *u e v* may preferably be laid in horizontal courses, while the part inside this line should have the joints radial. The top surface should be smoothed off and made sufficiently water-tight, so that it will drain off. A troweled surface of wet concrete makes a good finish.

EXAMPLES FOR PRACTICE

1. What depth of key is required for a concrete arch culvert of 12-foot span, with a rise of 2 inches per foot of span?

Ans. 1 ft. 8 in.

2. What is the radius of the intrados, and what the depth of key, for a circular arch culvert of cut stone, the span and rise being 10 and $3\frac{1}{2}$ feet, respectively?

Ans. $\begin{cases}\text{Radius, 5 ft. } 3\frac{7}{8} \text{ in.}\\ \text{Key, 1 ft.}\end{cases}$

3. What depth of key is required for a semicircular brick arch culvert having an 8-foot span? Ans. 1 ft. $2\frac{1}{2}$ in.

4. In a concrete arch culvert of 12 feet span and 12 feet clear height at the crown, and having a rise of 2 inches per foot of span, what are the dimensions of the cross-section of the abutments?

Ans. $\begin{cases} \text{Height, 10 ft.} \\ \text{Thickness at spring, 4 ft. } 2\frac{3}{8} \text{ in.} \\ \text{Thickness at base, 6 ft. } 8\frac{3}{8} \text{ in.} \end{cases}$

5. What is the thickness of the walls at the spring line and at the base for a circular arch culvert of cut stone, the span and rise being, respectively, 10 and $3\frac{1}{2}$ feet, and the height of wall 6 feet?

Ans. $\begin{cases} \text{At the spring, 3 ft. 5 in.} \\ \text{At the base, 4 ft. } 1\frac{1}{2} \text{ in.} \end{cases}$

PAVEMENT

61. A great many culvert failures are due to the undermining of the walls by the scour of the water rushing through. To avoid this danger, a pavement should be provided. Too little care in paving is common because its importance is not sufficiently realized; frequently, wedge-shaped or pyramidal stones are used, with the natural result that the pressure at the sides (or the upward pressure from the bottom if the soil is soft) dislodges them, destroying the pavement and eventually causing the ruin of the whole structure. Projections or irregularities in the upper surface may cause the lodging of drift and the clogging of the waterway, or give an opportunity for the drift to strike injurious blows.

The pavement should be made impervious, for, if water is allowed to pass through it, it is likely to find a channel underneath, either through voids already existing, or made by the softening or settling of the soil, and the scour proceeds as though no pavement had been laid. This danger is all the worse because it is out of sight and perhaps not realized. A casual inspection may reveal the trouble if a part of the pavement has been washed away, but underground watercourses may grow undiscovered.

A culvert may last a long time with a poor pavement or with no pavement, but the greater safety afforded by a good pavement is well worth its cost.

62. Cross-Walls.—As the velocity, and therefore the scouring action, of water is greatly diminished by a barrier that changes the direction of flow, **cross-walls**, also called **cross-sills,** are built at intervals under the culvert, extending from one side wall to the other. A cross-wall may frequently cause a cavity to fill up when otherwise a dangerous underground channel might form. Cross-walls are especially desirable if the culvert is long or has a steep grade. These walls should be made of large stone, with the top level with the top of the pavement, and their base at least 1 foot below the pavement.

63. Aprons and Apron Walls.—The danger of undermining is greatest at the ends of the culvert: at the upper end, before the water has been confined; and at the lower end, where the soil may be easily carried away by the stream. To prevent the scour from starting at these two points, the pavement is carried beyond the ends of the culvert at least as far as the wing walls extend, and farther if necessary for protection. This extension of the pavement is known as the **apron.** At the end of the apron, a cross-wall, called an **apron wall** or a **curb wall,** is built. If the apron is carried beyond the ends of the wing walls, an apron wall should be laid not only at the end of the apron, but also under the ends of the wings. If there is much fall to the ground surface below the mouth of the culvert, or if water stands about it, riprapping may be required for the safety of the culvert, as well as for the protection of the bank.

64. Stone Pavement.—Stone pavements should be not less than 12 inches deep. The stone should be sound, with nearly vertical sides, and nearly uniform in depth and transverse dimensions. Large stones are desirable for paving, but the allowable depth should not be reduced when they are used. The pavement should always be laid in mortar, or else it should be grouted after the stone is placed. This is more important for the pavement than for the walls and cover, especially when the foundation is soft or of a fine material.

Engineers differ as to the best design for pavements for stone box culverts: some prefer to excavate to a uniform depth and extend the pavement under the side walls; while others carry the walls deeper than the pavement, and pave only between the walls. The former plan makes the pavement an integral part of the structure, and reduces the danger of cracks being formed at the face of the walls where the walls join the pavement; when such cracks are formed, water runs through them and undermines the structure. The other design reduces the danger from frost, and frequently affords a firmer foundation.

65. Concrete Pavement.—In concrete culverts, especially in arch culverts, the pavement is usually made of the same material and with an invert; this pavement is readily made, and combines the advantages of the two plans described in the last article.

The form of the invert offers as much opportunity for variety as the roof arch itself—from a perfectly flat surface to a semicircular form. When the invert is made in the form of a circular arc, its thickness at the center may be determined by the formula used for an upright arch, by substituting the dishing of the invert for the rise h of the arch, and multiplying the result by from $\frac{1}{3}$, for a very firm foundation, to 1, for a soft foundation, according to the nature of the foundation. For an average foundation, multiply by $\frac{3}{4}$.

66. Considering the invert as a pavement, it seems wise, with a material so easily shaped as is concrete, so to lower the center below the sides that, when there is a very small volume of water, its course may be directed through the culvert instead of being collected in pools too shallow to run off, as might be the case if the pavement were flat. On the other hand, with a semicircular invert, or one with a large amount of dishing in proportion to the span, the waterway is reduced too much in its lower and first affected portion. A dishing of about 1 inch per foot of span seems to satisfy all requirements. The invert may consist of two straight slopes meeting at an angle at the center, as shown in Fig. 17, or of two

straight lines connected by a curve at the center, as shown in Fig. 16. In the former case, the slope of the pavement at the line of the face of the abutments is 2 inches per foot, while in the latter case it is at the rate of 4 inches per foot.

67. Cross-walls should be built under the invert of a concrete culvert, as indicated in Art. **62**, the same as if stone masonry were used. If the grade is steep, the foundations may be prepared in steps to give a succession of level bearings for the concrete, and the opening in the culvert may be made on an even grade; or the opening also may be stepped, as suggested in a subsequent article. Where practicable, the former method is preferable.

68. Timber Pavement.—The floor of timber culverts may be made by leveling off and compacting the soil and laying transversely on it timbers 6 or 8 inches thick. These timbers should be laid close; they should be made somewhat longer than the distance out to out between the walls of the culvert, and the first timbers of the walls should be drift-bolted to about every third floor timber.

An alternative arrangement that is somewhat less expensive but less durable is to place mud-sills at the toe of the slope at each side of the bank, and at intervals of about 6 feet—all with the tops on an even slope. These sills should be well and firmly bedded in the soil, and a smooth and firm surface made between them on the plane of the tops of the sills; on these sills is laid a floor of longitudinal plank 2 or 3 inches thick having the same width as the culvert, and being well spiked to the sills. The wall timbers are set outside this floor and drift-bolted to each sill. It is advisable so to space the sills that one will come under each joint in the first wall timbers, and each timber be drift-bolted to it.

CULVERTS WITH A GRADE

69. Culverts with a heavy grade may often be built with steps a few inches in height, having their tops slightly inclined in the direction of the grade. When this is done, the step in the cover should be made a little nearer (say about three times the amount of drop) to the lower end of

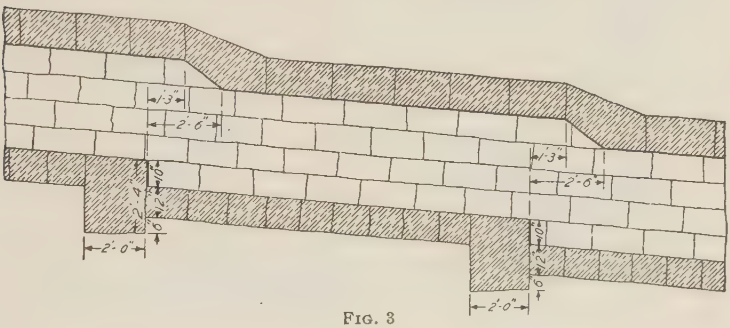


FIG. 3

the culvert than the step in the pavement; otherwise, the capacity of the culvert is reduced, and the danger of clogging is correspondingly increased. If the step in the cover can be relieved by using a beveled stone, this will reduce the danger of catching drift. A succession of steps from 6 to 10 inches in height is preferable to one large step

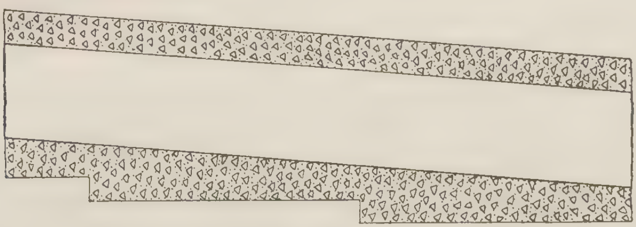


FIG. 4

of the same total drop. If the natural grade of a stone culvert exceeds about 10 per cent., it is preferable to use steps in this way, so that the bed of each stone may be laid on the bottom with comparatively little inclination, and be therefore in no danger of sliding. Wherever such steps are

introduced, cross-walls should be used; these walls should be at least 2 feet deep and extend 6 inches below the bottom of pavement at the foot of the step. Fig. 3 shows a longitudinal section of a stone culvert built on a grade.

If the culvert is of concrete, the same method may be adopted, but it is preferable, where possible, to leave the opening of the culvert on a uniform grade and step off the foundations to give a succession of level bearings, as shown in Fig. 4.

70. If the foundation of a culvert is on rock in which the natural grade exceeds 10 per cent., advantage may be taken of this natural pavement; but that part of the rock that supports the walls should be dressed off to give approximately a level bed for each stone resting on it, and the walls should be laid in nearly horizontal courses, the cover being stepped down at intervals as may be required.

WING WALLS AND PARAPETS

71. Object of Wing Walls.—As the covered part of a culvert is rarely extended as far as the toe of slope of the bank, **wing walls**, or **wings**, are built at each end to prevent the fill from running into or across the mouth of the culvert. The wings serve also to direct the course of the water into or away from the culvert so as to prevent its getting behind the main walls.

72. Kinds of Wing Walls.—Wing walls are divided into *straight wings*, *flaring wings*, and *transverse wings*. **Straight wings** are continuations of the culvert walls in the same direction as the body of the culvert; **flaring wings** are turned back at an angle with the culvert; **transverse wings** are at right angles with the culvert, or are parallel with the bank if the culvert is skewed.

73. Straight wings are generally used for stone box and for timber culverts, unless the direction of the watercourse demands another form of wing. Straight wings are the

most economical, and are ordinarily stepped down according to the slope of the bank (see Fig. 5).

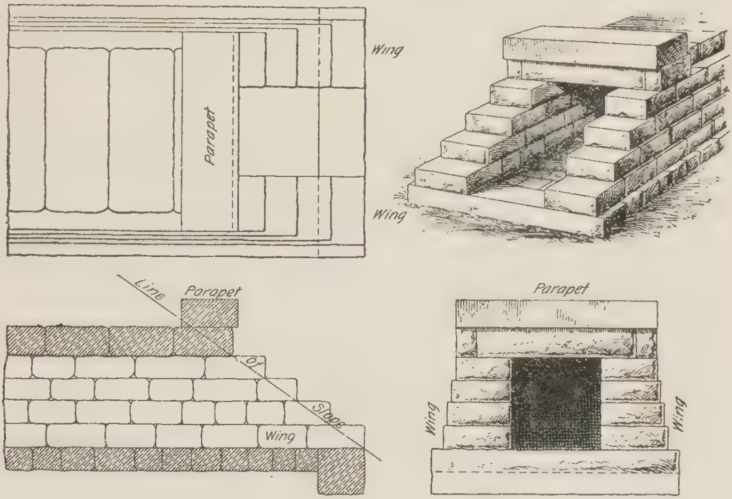


Fig. 5

74. Flaring wings naturally accompany arch culverts, and also concrete culverts, even if the cover is flat. The

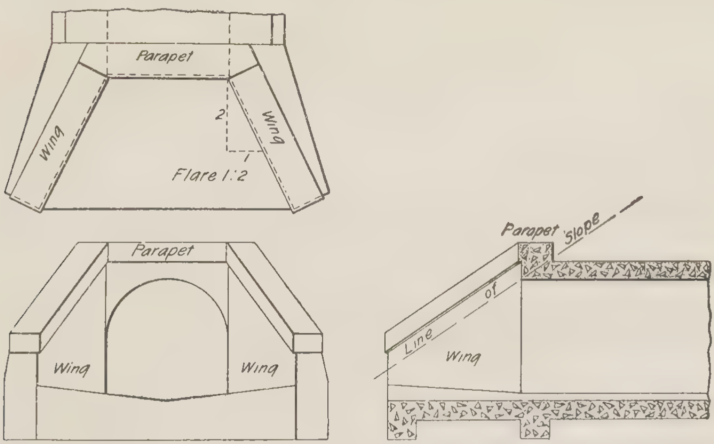


Fig. 6

enlarged inlet and outlet they form aid in increasing the capacity of the culvert, by easily directing the water into it.

and allowing the discharge to spread and so decrease in depth before leaving the line of the bank and the protection given by the pavement (see Fig. 6). The amount of **flare,** or **splay,** as the deviation of the wings from the direction of the culvert walls is called, need not be very great; a flare of 1 : 2 (see Fig. 6) is generally sufficient. With a large flare, the length of wing and the quantity of masonry are increased with no corresponding gain in efficiency.

As with straight wings, flaring wings may be stepped down or sloped to correspond with the bank they restrain.

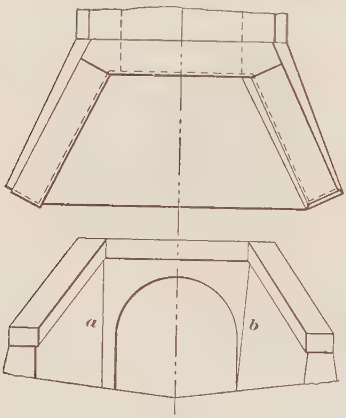


FIG. 7

In joining the wings with the abutments or end walls, there should be no offset made in the face of the masonry, for such jogs will cause eddies, and so retard the water in its flow, besides adding materially to the expense; to this must be added that, at the upper end, they will facilitate the accumulation of drift. The junction should be as shown in Fig. 6, not as shown at *a* or at *b*, Fig. 7. For this reason, the faces of wing walls should not be battered, unless either the end wall or the abutment inside the culvert is also battered. A vertical face of the side wall is sometimes continued beyond the ends of the cover, and is intersected with a battered face of the wing outside the culvert; but this adds to the expense of the forms without any corresponding advantage, the supposed esthetic superiority being the only excellence claimed.

75. Transverse wings are sometimes used for either stone or concrete culverts, whether of box or arch form, and are the common form for pipe culverts. They are carried to a uniform height for their whole length, and the slope is allowed to run around in front, the length being determined

by this slope. Because of the ends of the wings being covered with the slope, there is less danger of their being undermined than with straight or flaring wings. Fig. 8 illustrates the transverse wings.

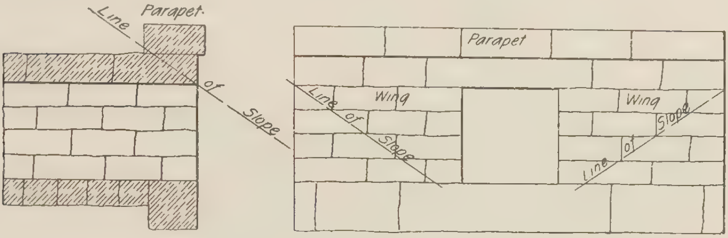


FIG. 8

76. Parapets.—The length of stone or concrete culverts is generally reduced by building **parapet walls** across the line of culvert at the end of the cover, extending the full width of the culvert and side walls, or the whole length of the wings if transverse wings are used. For appearances, the top course of the parapet is generally made to project from 2 to 4 inches beyond the face of the cover and lower courses (if any) of the parapet (see Figs. 5, 6, and 8).

FOUNDATIONS

77. The principles governing the foundations of culverts are the same as for other foundations; but, on account of the large areas covered by culverts in proportion to their expense, it is seldom that the same care is warranted in preparing the foundation that would naturally be taken for a bridge pier or abutment. When the soil is gravel or a moderately firm earth, it is customary to excavate only as required for the pavement and the walls; but, if the bottom is swampy, a good foundation may be formed by timbers or logs laid longitudinally and transversely. *Timber should not be so used unless the ground will always remain damp*, for otherwise the timber will rot, and the foundation will be worse than if no timber had been used. If the soil is soft and compressible, but unsuitable for the use of timber, it may be excavated to a greater depth, and made more firm and

compact by tamping sand, gravel, or broken stone into the softer material.

78. If the culvert rests on rock, the pavement is omitted, but all rotten stone and loose scale should be removed, and the surface prepared so as to give a good level bed for the masonry. If the soil is partly rock and partly softer material, great care must be taken to avoid settlement and a break in the culvert at the point of change, and also to make a safe joining of the paving to the rock.

79. When the foundation is soft or varies greatly in character, there is danger of transverse breaking or distortion of the culvert by the uneven settlement. The writer has seen a culvert that was built on a uniform grade under a high bank across soft ground which had settled so much that one could not see from end to end, and water was standing nearly to the roof at the middle when both ends were dry. Where settlement is expected, the middle of the culvert may be laid higher than the ends; but no empirical rule can be given for the amount of settlement: judgment founded on individual experience is the only satisfactory guide.

LENGTHS

80. The length of a culvert is governed by the height of the bank and the width of the roadbed, and by the style

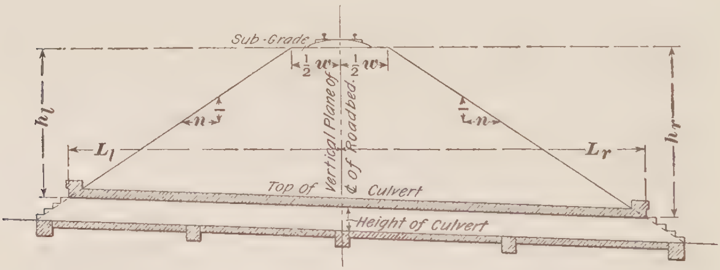


Fig. 9

of end to be used. Generally, it is so chosen that the plane of slope of the bank shall come just to the top of the culvert at the end of the cover.

As the two ends of a culvert are seldom at the same level, the length of each side must be determined separately. The following formulas may be used (see Fig. 9):

$$L_l = \frac{w}{2} + n\,h_l \qquad (1)$$

$$L_r = \frac{w}{2} + n\,h_r \qquad (2)$$

in which

L_l and L_r = horizontal lengths from center line of roadbed to end of cover, on the left and right sides, respectively, measured at right angles to the roadbed;

w = width of roadbed at subgrade;

h_l and h_r = difference in elevation between subgrade and top of culvert at the left and right sides, respectively;

n = slope ratio (horizontal distance divided by corresponding vertical distance) of side of bank (commonly $1\frac{1}{2}$, but may be more if the bank is composed of sand or clay, or less if of large loose rock).

If it is desired that the length to the ends of the wings should just reach the toe of the slope, L may be considered as the length to, and h the height of the bank above, this point, instead of referring to the end of the cover. If the culvert is on a skew with the bank, the center line and subgrade elevation opposite the end of the culvert must be used instead of that over the axis of culvert. If the parapet and wings are to be built normal to a skew culvert, allowance must be made for the skew in the width of the structure. When the culvert has a heavy grade, the wings should be lengthened at the lower end, and may be shortened at the upper end.

Example.—What is the horizontal length of cover from the center line of an 18-foot roadbed for a culvert 4 feet high, the elevation of subgrade being 117.2 and that of the bottom of culvert at the end being 94.7, the slope of bank being 1.5 : 1?

Solution.—Here, $w = 18$, $n = 1.5$, and $h_r = 117.2 - 94.7 - 4 = 18.5$. Substituting in formula 2,

$$L_r = \frac{18}{2} + 1.5 \times 18.5 = 36\tfrac{3}{4} \text{ ft.} \quad \text{Ans.}$$

DETAILS AND ILLUSTRATIONS OF CULVERT CONSTRUCTION

PIPE CULVERTS

81. Pipe culverts, whether of terra cotta or of cast iron, are sometimes laid with no other preparation than simply hollowing out the soil to receive the pipe, or excavating a trench and filling it with sand up to the middle of the pipe. If the soil is firm, the former method may prove satisfactory when proper care is taken to tamp well the filling required to replace the excess of material that is usually removed. Sand packed around the pipe makes an excellent bed if undisturbed, but even a slight leak in the pipe may wash away the sand. The best practice is to lay the pipe in a bed of concrete extending up to the middle of the pipe.

82. Whether the pipe is set in a concrete bed or rests directly on the soil, all joints should be calked or laid in cement. The pipe should always be laid with the bell end upstream. If practicable, there should be sufficient grade in

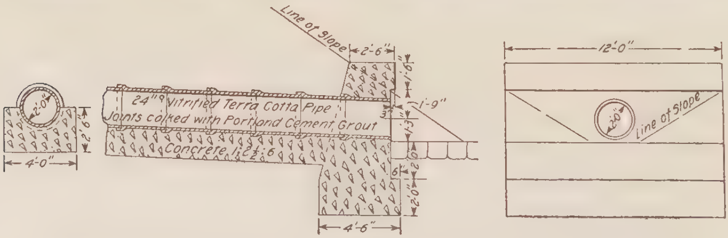


FIG. 10

pipe culverts, especially at the lower end, to prevent settlement from forming pockets in which water may collect and freeze. When the ground is soft and only a slight fall is possible for the whole length of culvert, it is wise to raise the central part of the culvert even above the inlet, as the

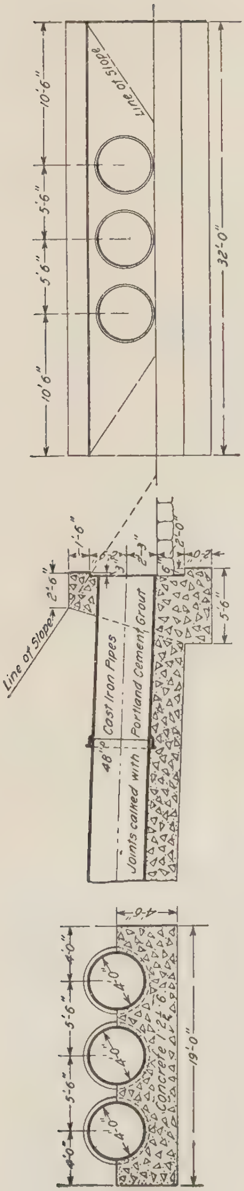


FIG. 11

impounding of a small quantity of water at the upper end of the culvert is less serious than to allow water to collect within the pipe. The ends of the pipes should be protected by head-walls of concrete or stone masonry, which should be long and high enough to prevent obstruction of the mouth of the pipe by the wash of the bank. The head-walls should be carried to such a depth as to avoid the dangers from frost and undermining. With pipe culverts, liberality in the length of the culvert may effect a saving in the cost of head-walls equal to many times the extra cost of pipe. Unless the flow through a pipe culvert is expected to be small, a pavement should be laid below the outlet, or riprap should be used.

Fig. 10 shows a single terra-cotta pipe culvert, with 24-inch pipe, and Fig. 11 shows a triple cast-iron pipe culvert with 48-inch pipe.

83. Pipe culverts are used largely to replace decayed timber culverts. The timber flooring may be removed, and even the walls and roof may be partly cut away for the brief time required for the work, so that pipe of as large an inside diameter as the smaller dimension of the timber culvert may readily be substituted without interfering with the bank. The space outside of the pipe should be packed earth and spalls. This method of building temporary culverts of

timber and replacing them later with pipe is especially satisfactory, because during the life of the timber the bank will settle and the engineer can see whether his estimate for the size of the opening was correct. Timber culverts may be made larger than absolutely required, so as to allow pipe of the estimated capacity to be readily substituted when the time for renewal arrives.

TIMBER BOX CULVERTS

84. In timber culverts, as the walls are not designed to resist the thrust by their weight, the timbers should be thoroughly drift-bolted together (see Art. **44**). If the pavement is made of a succession of thick floor timbers laid close, the first timbers of the side walls should be bolted to the floor timbers about every 3 feet; or, if a thin floor is laid on sills, two bolts should be used at each end of each sill. The wall timbers should be butt-jointed and laid to break joints not less than every 4 feet, and have at least two drift bolts between adjacent joints. The different courses of wall timbers should be bolted together at intervals of about 5 feet, the bolts being long enough to reach two-thirds through the next lower timber.

85. The cover is made of transverse timbers laid close together, and of a thickness determined as stated in Arts. **37**,

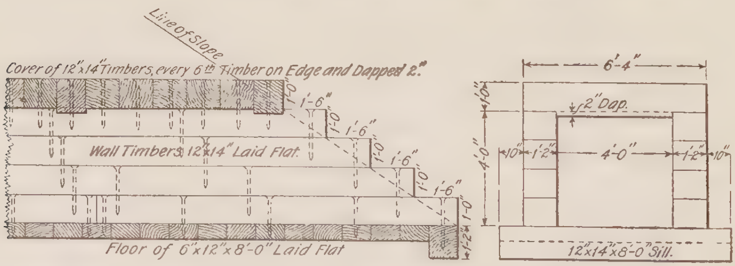


Fig. 12

38, **39**, and **42**. Each of these cover timbers should be bolted to the top timber of the side walls, and every sixth or eighth cover timber should be notched down on the walls.

Timber culverts are commonly built without head-walls or parapets, but the ends of the side walls are stepped to serve as wings to retain the slope. Fig. 12 shows a typical timber box culvert 4 feet square.

STONE BOX CULVERTS

86. The cover and walls of a stone box culvert are proportioned as explained in Arts. **37** to **45**, inclusive. Their faces should be so smooth that there will be no danger of drift lodging or striking so hard as to dislodge a stone. The cover stones are generally laid with less care as to close joints than is customary for wall masonry, for their office is chiefly to act as beams to carry the bank above, and to some extent each stone does its work independently of the rest. In order, however, to make the cover tight, the openings between the stones should be filled with spalls laid in mortar.

87. The thickness of the walls at top and bottom, as determined in Art. **44**, is the minimum to be allowed, but it is not necessary that the back of wall should be dressed to a battered line; generally, the back will be rough, and the wall may be made as thick at the top as it must be made at the bottom. In estimating quantities for the contractor, however, a uniform slope on the back is used, as the quantity within this plane is what is absolutely required. No course or stone on the back of the wall should be allowed to overhang the one below.

88. A cross-section, a longitudinal section at the end, and an end view of a typical single box culvert of stone are shown in Fig. 13; and for a double box culvert, in Fig. 14. In the latter figure, the longitudinal section shows the middle wall; the side walls are the same as for a single box culvert. The dimensions of the different parts for these culverts are given in Table II, in which the dimensions not given for double culverts are the same as for single culverts. These dimensions are suitable for sandstone or limestone of good quality. The letters given in the table refer to Figs. 13 and 14.

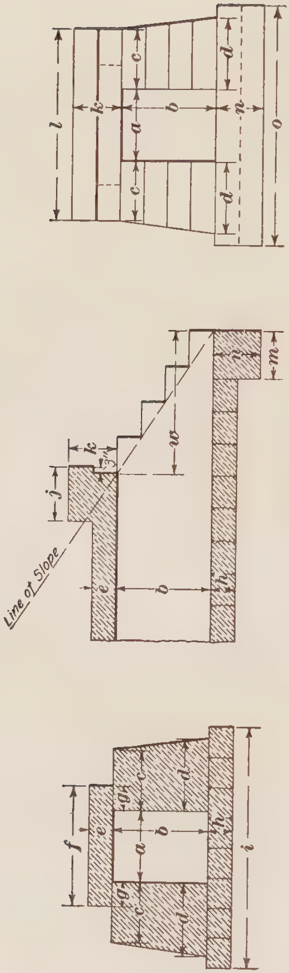


Fig. 13

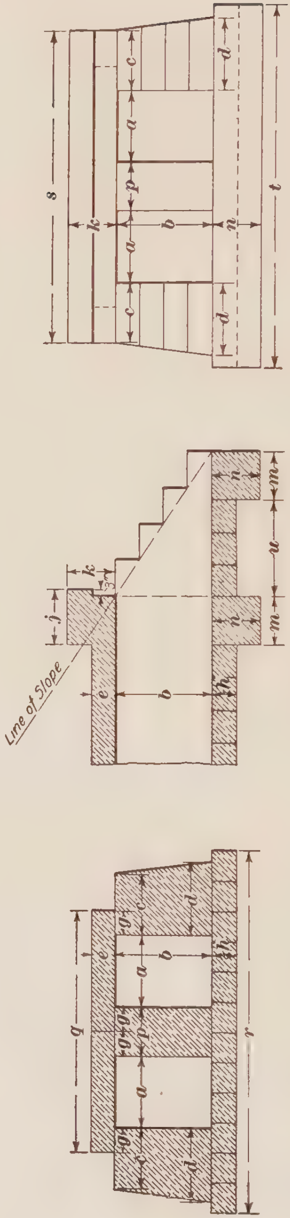


Fig. 14

TABLE II

DIMENSIONS FOR STONE BOX CULVERTS

Size of Opening			Single Culverts																											Double Culverts											
			Thickness of Side Walls				Cover Stone						Pavement				Parapet						Apron Wall					Wings		Dimensions the Same as for Single Culverts, Except as Given Below											
Width *a*		Height *b*	At Top *c*		At Bottom *d*		Depth *e*		Length *f*		Bearing on Wall *g*		Depth *h*		Width *i*		Width *j*		Height *k*		Length *l*		Width *m*	Depth *n*		Length *o*		Length *w*		Thickness of Middle Wall *p*		Width of Cover *q*		Width of Pavement *r*		Length of Parapet *s*		Length of Apron Wall *t*		Between Apron Walls *u*	
Feet	Inches	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches
2		3	2		2	6		9	4		1		1		8		1	9	1	6	6		2	2		8		4	6												
2	6	3	2		2	6		10	4	6	1		1		8	6	2		1	9	6	6	2	2		8	6	4	6												
3		3	2	3	2	9	1		5		1		1		9	6	2	3	2		7	6	2	2		9	6	4	6	2		10		14	6	12	6	14	6	2	6
3		4	2	6	3		1		5		1		1		10		2	3	2		8		2	2		10		6		2		10		15		13		15		4	
3		5	2	9	3	6	1		5		1		1	2	11		2	3	2		8	6	2	2	2	11		7	6	2	6	10	6	16	6	14		16	6	5	6
4		4	2	6	3		1	4	6	8	1	4	1		11		2	6	2	8	9		2	2		11		6		2	8	13	4	17	8	15	8	17	8	4	
4		5	2	9	3	6	1	4	6	8	1	4	1	2	12		2	6	2	8	9	6	2	2	2	12		7	6	2	8	13	4	18	8	16	2	18	8	5	6
4		6	3		4		1	4	6	8	1	4	1	4	13		2	6	2	8	10		2	2	4	13		9		3		13	8	20		17		20		7	

89. Another design for stone box culverts is shown in Fig. 15. This form is preferred by many engineers; but, in the majority of cases, the form shown in Fig. 13 seems to be preferable, for the reasons stated in Art. **64.** The

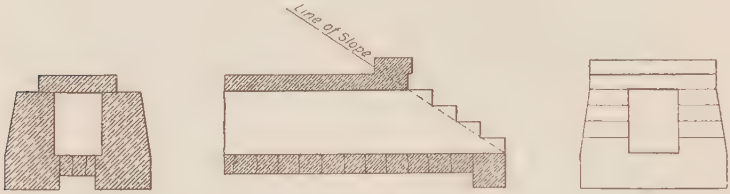


Fig. 15

dimensions for the two designs would be the same, except for the arrangement of the paving and the bottom part of the walls. Double culverts are not used for openings less than 3 ft. × 3 ft.

STONE ARCH CULVERTS

90. Stone arches are rarely desirable for culverts with small openings, because this style of construction is too expensive for spans where ordinary stone covers are practicable. In places, however, where the appearance is important in determining the design, stone arches may be used, even though the span is small.

91. In the main, the design of stone arch culverts is similar to that of concrete arches; but, owing to the great cost of cutting the arch stones, a flatter arch would be suggested for stone than for concrete. The ratio of rise to span may be made from 1 : 3 to 1 : 6 for stone arches, according to the prices for material and work.

CONCRETE ARCH CULVERTS

92. The concrete arch seems destined to become the common type of culvert, even in localities abounding with good stone. Many engineers batter the inner surface of the side walls of concrete arch culverts below the spring lines, but, unless the span of the arch is increased, this reduces the

width of waterway at the bottom, with the result that the total capacity of the culvert is decreased, while the cost is slightly increased.

93. Though in stone box culverts there is little attempt made to protect the structure from the effect of frost, in concrete arches the foundation is commonly carried below the frost line, for the effect of frost may impair the strength of the structure by destroying its monolithic character. The frost line being deeper at the ends of the culvert than where the culvert is partly protected by the bank above, there is greater need of deep foundations under the wings and apron than under the body of the culvert.

94. Fig. 16 shows a typical semicircular concrete arch culvert. The dimensions required for culverts of this form may be found in Table III. Fig. 17 and Table IV give arches with a rise of one-quarter of the span. In the culvert shown in Fig. 16, the invert is curved, while in that shown in Fig. 17 the invert has straight sloping sides. Either style of invert is applicable to both forms of arch.

A comparison of the dimensions for these two forms shows that, for any given span and height of culvert, there is little difference between the semicircular arch and one with a rise of one-quarter of the span (or even one-sixth of the span), either in the waterway or in the required depth of key and thickness of the abutment at the spring line of the arch. As the flattening of the arch raises the spring line, and also increases the rate of spread of the walls, there is a decided increase in the quantity of masonry as the arch is flattened; the maximum of economy and width of structure favors the semicircular arch. For small concrete arch culverts, the most economical design is that in which the height is about equal to the span.

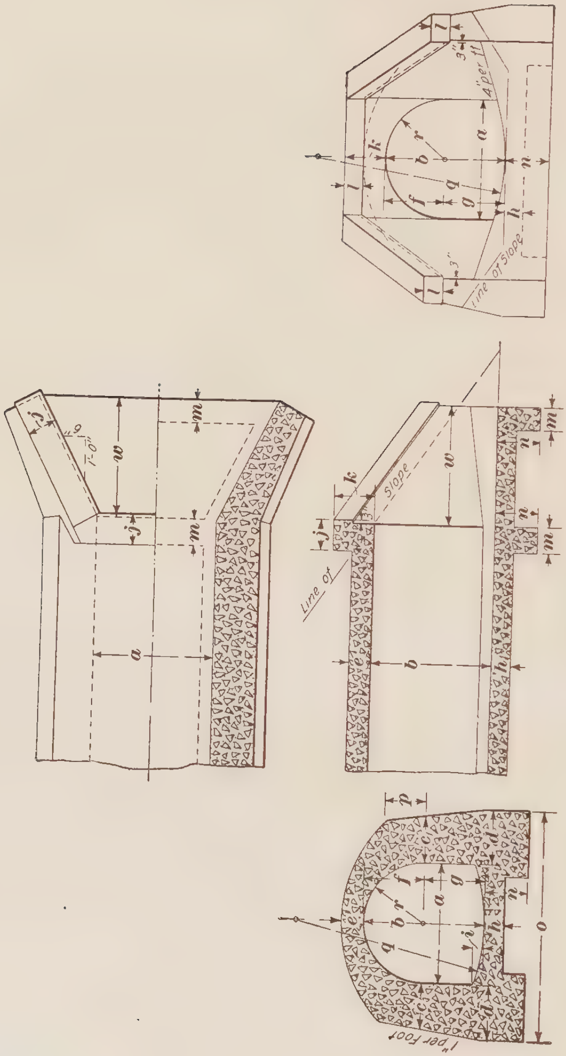


FIG. 16

TABLE III

DIMENSIONS FOR CONCRETE SEMICIRCULAR ARCH CULVERTS

(Portland-Cement Concrete: for Arches, 1 : 2 : 4; for Walls, 1 : 2½ : 6)

Size of Opening: Width a	Size of Opening: Height b	Thickness of Abutments: At Spring Line c		Thickness of Abutments: At Bottom d		Depth of Key e		Rise of Arch f		Height of Wall to Spring Line g		Thickness of Invert h		Dip of Invert i		Width of Parapet and Coping of Wings j		Height of Parapet k		Depth of Coping on Parapet and Wings l		Width of Apron and Cross-Walls m	Depth of Apron and Cross-Walls n	Total Width o		Height of Back of Walls Above Spring Line p		Radius of Invert q		Radius of Crown r		Length of Apron w	Area of Opening x
Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Square Feet
3	3	2	6	2	7½	1	1	1	6	1	6	1	1		3	2	6	2	7	1	6	2	4	8	3	1	3½	4	7½	1	6	3	7.8
4	4	2	8	2	10	1	2	2		2		1	2		4	2	6	2	8	1	6	2	4	9	8	1	7	6	2	2		4	13.8
5	5	2	10	3	1½	1	3	2	6	2	6	1	3		5	2	6	2	9	1	6	2	4	11	1	1	10½	7	8½	2	6	5	21.6
6	6	3		3	3	1	4	3		3		1	4		6	2	6	2	10	1	6	2	4	12	6	2	2	9	3	3		6	31.1
8	8	3	4	3	8	1	6	4		4		1	5		8	2	6	3		1	6	2	4	15	4	2	9	12	4	4		8	55.4
10	10	3	8	4	1	1	8	5		5		1	8		10	2	6	3	2	1	6	2	4	18	2	3	4	15	5	5		10	86.5
12	12	4		4	6	1	10	6		6		1	10	1		2	6	3	4	1	6	2	4	21		3	11	18	6	6		12	124.6

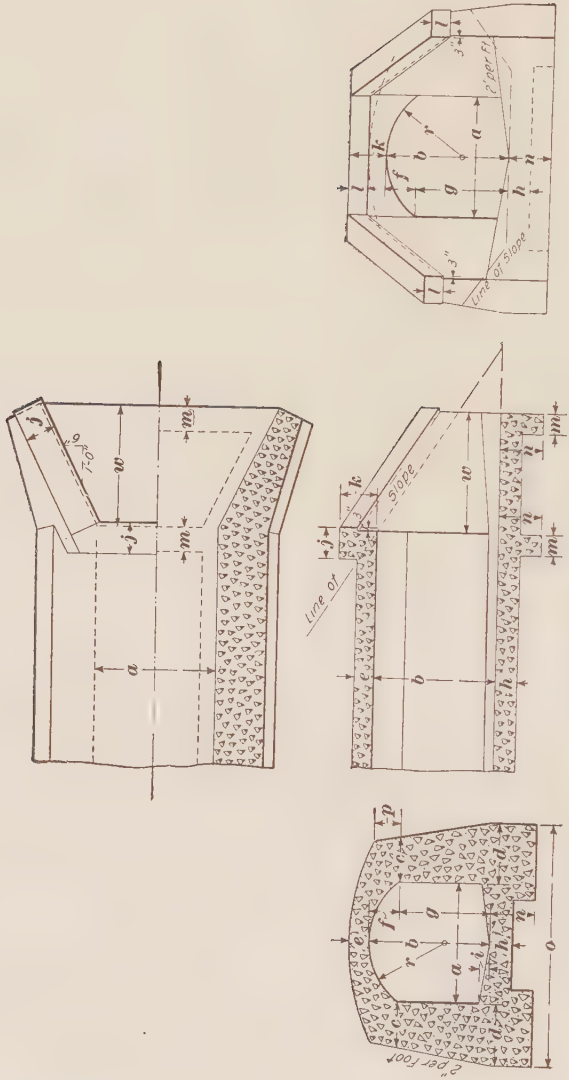


FIG. 17

TABLE IV

DIMENSIONS FOR CONCRETE ARCH CULVERTS WITH RISE = $\frac{1}{4}$ SPAN

(*Portland-Cement Concrete: for Arches, 1 : 2 : 4; for Walls, 1 : 2½ : 6*)

Size of Opening		Thickness of Abutments																													
Width *a*	Height *b*	At Spring Line *c*		At Bottom *d*		Depth of Key *e*		Rise of Arch *f*		Height of Wall to Spring Line *g*		Thickness of Invert *h*		Dip of Invert *i*		Width of Parapet and Coping of Wings *j*		Height of Parapet *k*		Depth of Coping on Parapet and Wings *l*		Width of Apron and Cross-Walls *m*	Depth of Apron and Cross-Walls *n*	Total Width *o*		Height of Back of Walls Above Spring Line *p*		Radius of Crown *r*		Length of Apron *w*	Area of Opening *x*
Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Feet	Feet	Inches	Feet	Inches	Feet	Inches	Feet	Square Feet
3	3	2	6	2	10½	1	1		9	2	3	1	1		3	2	6	2	7	1	6	2	4	8	9		11	1	10½	3	8.1
4	4	2	8	3	2	1	2	1		3		1	2		4	2	6	2	8	1	6	2	4	10	4	1	1	2	6	4	14.4
5	5	2	10	3	5½	1	3	1	3	3	9	1	3		5	2	6	2	9	1	6	2	4	11	11	1	3	3	1½	5	22.4
6	6	3		3	9	1	4	1	6	4	6	1	4		6	2	6	2	10	1	6	2	4	13	6	1	5	3	9	6	32.3
8	8	3	4	4	4	1	6	2		6		1	6		8	2	6	3		1	6	2	4	16	8	1	9	5		8	57.4
10	10	3	8	4	11	1	8	2	6	7	6	1	8		10	2	6	3	2	1	6	2	4	19	10	2	1	6	3	10	89.7
12	12	4		5	6	1	10	3		9		1	10	1		2	6	3	4	1	6	2	4	23		2	5	7	6	12	129.2

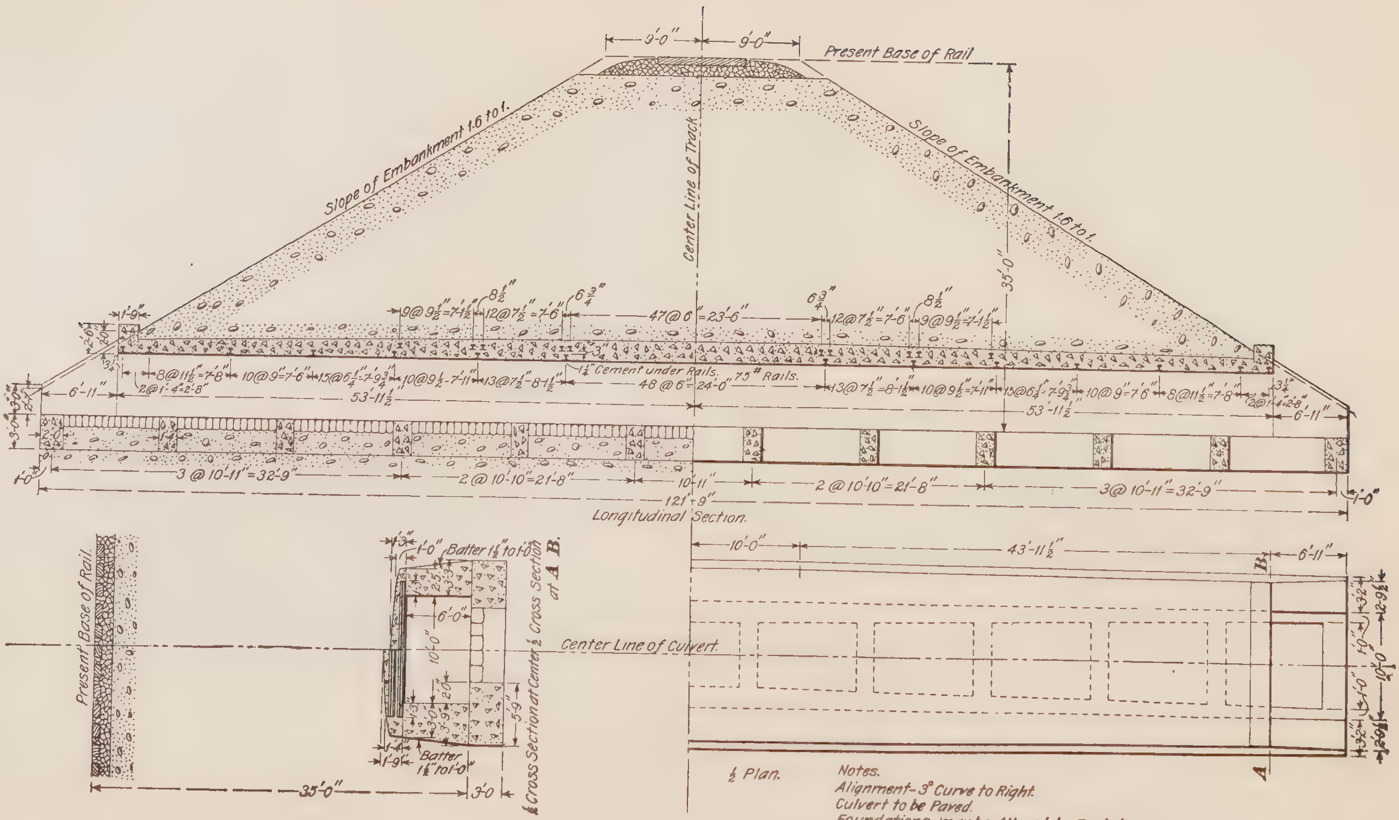

Present Base of Rail
Slope of Embankment 1.6 to 1.
Center Line of Track
1¼" Cement under Rails.
75# Rails.
Longitudinal Section.
½ Cross Section at Center ½ Cross Section at A B.
Center Line of Culvert.
½ Plan.
Notes.
Alignment-3° Curve to Right.
Culvert to be Paved.
Foundations may be Altered by Engr. in Charge.

FIG. 18

FLAT-TOP REINFORCED-CONCRETE CULVERTS

95. Concrete box culverts, in which the concrete in the cover is reinforced with steel, seem destined to be very extensively used in the future. A great number of culverts of this class have been built by the Chicago, Milwaukee & St. Paul Railway, in which the reinforcement is made of old rails or I beams. One of these culverts, 10 feet by 6 feet, is shown in Fig. 18, taken from a paper read in 1901 by Albert Reichmann before the Western Society of Civil Engineers.

It will be noticed, in this culvert, that underneath the track and on each side of the center line, for a distance about equal to the height of the bank above the cover of the culvert, two tiers of rails are used; and that for a width of about 24 feet at the center they are placed as close together as they can well be laid and still leave room for any concrete between them. Where the height of bank is less, only one tier of rails is used, and the spacing is increased with the decrease of the depth of fill. A reinforcement of the cover with I beams is used in the same way as is shown for rails, and will allow the use of larger spans.

96. Rods also, either plain or in some of the patented forms, are used, as indicated in Fig. 19. The ends of some of the rods are turned up to carry the shear, or short inclined rods may be attached to the main horizontal rod. The longitudinal rods are introduced in the invert to strengthen the culvert for resisting the longitudinal bending moments due to uneven settlement; they may be omitted if the foundation is firm. Where the length desired to be reinforced in either direction exceeds the length of the rods, it is customary to simply lap them over each other a few inches without attempting any other splice than the concrete gives; or the ends of the rods may be looped back to get a better hold on the concrete.

97. Expanded metal or wire netting is similarly used (see Fig. 20), and has the advantage of being already put

together; while with rods each piece must be handled separately and held in place during construction. By the use of rods, however, the steel may be distributed more advantage-

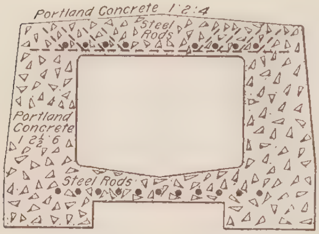


FIG. 19

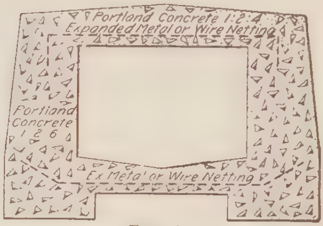


FIG. 20

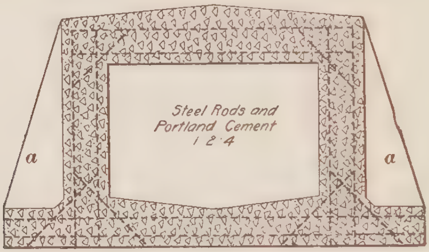


FIG. 21

ously. Rods or netting are sometimes used in the walls, as shown in Fig. 21, with buttresses a, a to stay the wider base to the walls.

REMARKS ON CONCRETE WORK

98. For culverts, it is customary to make the excavations for the foundations of walls and pavement (including apron walls and other cross-walls) of just the shape and dimensions desired for these parts; the concrete is tamped into these pits, completely fills them, and takes their form. The top of the invert is shaped by a board drawn over it, a thin layer of rich mortar being added and the top surface of the concrete being finished off by sprinkling on dry cement, which will absorb enough of the moisture from below to acquire the proper consistency. Under where the walls are to be built, the surface of the concrete is left rough; and, unless

the forms can be placed and more concrete added before the lower layer sets, it is advisable to place large irregular stones partly embedded in and partly projecting above the top of the concrete, to serve as dowels to bind the new to the old concrete when the work is resumed. While the foundations are being laid and the invert is being finished off, forms and the bracing for them should be prepared in sections of about 10 or 12 feet, so that they can be quickly put in place. If possible, the work of placing the concrete should be resumed before concrete previously laid has had time to take its initial set.

If practicable (and in culvert work it is rarely otherwise), the forms should be so designed that they can be held rigidly in place without rods or any ties passing through the concrete.

99. Fig. 22 shows the cross-section of a concrete arch culvert with the forms in place. The plank lining (called **lagging** or **sheeting**) is usually made of $1\frac{1}{4}$- or $1\frac{3}{4}$-inch plank. The frames of the lower portions and the bracing are of

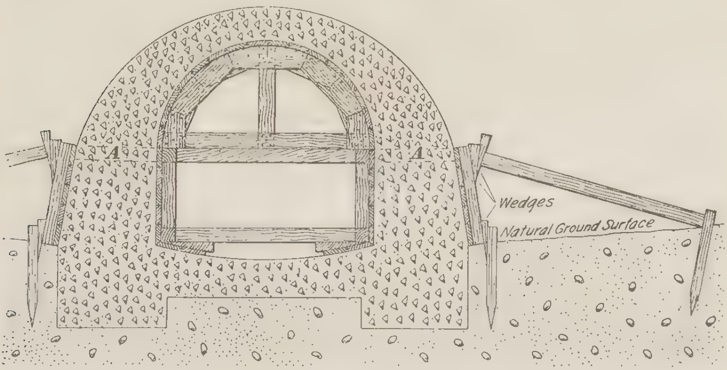


Fig. 22

joists from 4 in. × 4 in. to 6 in. × 6 in., according to the size of the culvert. The ribs for the arches are usually made of 2-inch plank well spiked together, with horizontal ties and braces of the same; the ribs are spaced according to the load to be carried, and additional braces may be

added if the span requires it. The whole inside form is laid on lagging placed on the invert (which, because of the mode of finishing its surface, hardens more rapidly than the sides under the walls) with wedge-shaped fillers between. The **centers** (as the forms for the arch are called) may be temporarily braced, as indicated by the broken lines at *A A*, until the concrete has reached nearly to that height, when the braces may be removed and the concrete will prevent the moving of the centers. The outer forms may be braced against rocks, if there are any conveniently located, or large stakes may be driven into the ground, and braces footed against them. The lagging should be tongued and grooved, or grooved at both edges and laid with splines, and planed on that side at least against which the concrete is to be laid; it should be matched so true that the outer surface will be smooth. The bracing should be spiked to the forms sufficiently to prevent falling apart, but left so that they can readily be knocked loose when the concrete has set and the forms are to be removed. The top of the structure is finished off with a trowel. In a flat-top culvert, the top should be sloped slightly to drain off any water seeping down through the bank.

Brushing the face of the lagging with soft soap will improve the appearance of the concrete. The lagging is sometimes covered with tin for the same purpose; but as this is expensive, and culverts seldom require much in the way of appearances, such covering may be advantageously dispensed with.

100. There has been a great difference of opinion among engineers as to the amount of water to be used in mixing concrete; but it is now generally conceded that a wet concrete will leave a harder and neater surface, and be more nearly water-tight than a dry mixture, especially if the lagging has been soaped, as suggested in Art. **99.**

When concrete work has to be suspended for so long a time that the old concrete sets before new is added, it is advisable to provide stone dowels, as stated in Art. **98;**

but, whether they are used or not, the old surface should be roughened up and thoroughly wetted before laying new concrete, so as to give the proper bond, and a layer of mortar of neat cement should be plastered on before concreting is resumed.

STANDARD PLANS

101. It is not customary to make detail plans of each culvert, but a **standard plan** is made at the central office for each size of culvert opening; or a general plan is prepared, with the various dimensions represented by letters or other characters, accompanied by a table giving, for each size of culvert contemplated, the figures for each part represented (see Figs. 13, 14, 16, and 17, with their accompanying Tables II, III, and IV). These standard plans usually show simply the cross-section, longitudinal section (and perhaps plan) of the end, and an end view; and may also have directions about modifications to be made for varying conditions. Even though the rules are very explicit, there may be nearly as great a call for good judgment on the part of the resident engineer in these modifications as in determining the location and size of the opening required.

LAYING OUT

102. The required waterway having been determined, the ground should be examined to see whether a good foundation can be obtained for the culvert at the lowest point of the embankment, or whether a decidedly better foundation can be found at a point slightly higher, but within the limit to which it is allowable to impound the water (Art. 3). The natural watercourse may not be exactly at right angles with the line of the bank, but, if practicable, it is advisable that the culvert should be so placed. If, however, the cost of grading the inlet and outlet will exceed the saving in masonry, more will be lost than gained by squaring up the line of the culvert.

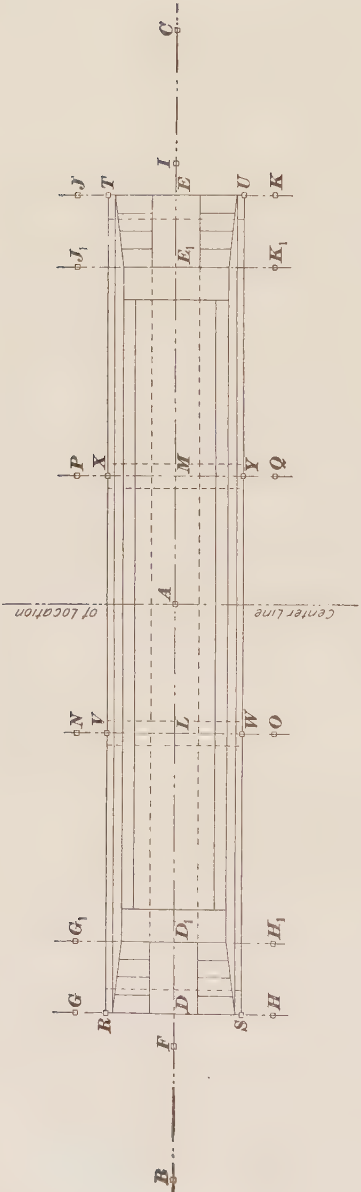


FIG. 23

103. The position and direction of the culvert having been settled, the center line should be marked on the ground by a stake set at the center line *A*, Fig. 23, and one on each side at such points *B* and *C* that they will be safe from disturbance, and so located that all parts of the site may be seen through a transit set over either stake. Next, noting from the plan the relation of the ends of the structure to the slope line, stakes are set at *D* and *E*, where the ends of the paving should be, as determined by the formulas in Art. **80.** From *D*, stakes *F*, *G*, and *H*, in range with the center line of the culvert and at 90° to it, are set at such distances from *D* that they will not be disturbed by the excavation for foundations, and yet will be close enough to be convenient for the use of the workmen. Similarly, from *E*, stakes are set at *I*, *J*, and *K*.

Having ascertained the length of the culvert, the number of cross-sills is readily determined and

their location fixed, as at L and M, and stakes N, O, P, and Q are set to mark the range of their axes. It is advisable, if practicable, to place the stakes $F, G, H, I, J, K, N, O, P$, and Q all at the same distance (say 3 feet) from the edges of the foundation. All these stakes should be firmly driven, and the exact range marked on them by tacks; each stake should be marked to indicate plainly for what it is intended to give line; as, ℄, or *C. L.* (center line) for stakes F and I; *End* for G, H, J, and K; and *X wall* for N, O, P, and Q. The height of the top of the pavement opposite, above, or below the level of the top of the stake should also be marked on the side. If desired, special grade stakes may be set to give the level of the pavement; if such special stakes are set, they should be cut to a different shape from those used for ranges. Stakes should also be set at R, S, T, U, V, W, X, and Y, at the edges of the excavation, and left high so that they may be readily seen; these stakes have no tacks, and may be displaced as soon as the excavation is begun.

Some foremen prefer to have the ends of the cover staked out, and make their own allowance for the wings, instead of having stakes for the ends of wings and making allowance for the cover. In this case, stakes are set at G_1, H_1, J_1, and K_1, instead of at G, H, J, and K.

ESTIMATE

104. As culverts are usually built, the cost, and sometimes also the contract price, for the pavement of stone culverts is less than for the walls and cover. The parapets and wings, or perhaps simply the coping stone, are also frequently classed separately from the rest of the culvert. If the culvert is founded on rock for the whole or a part of its length, there need be no other pavement, but all loose and scaly rock must be removed, and the surface of the rock must be leveled off to give a suitable bearing for the walls. The surface of the rock in such cases is usually so irregular that the height of the wall will vary considerably; and, the capacity of the culvert being limited by its minimum opening,

these varying heights increase the amount of masonry in the walls. Standard plans are generally accompanied by a schedule of the quantity of masonry of each class per running foot of culvert, with a stated addition for the parapet, wings, and apron at each end. It is customary with some engineers to make estimates for the contractors from these figures, disregarding the changes that may be caused by the procedure; in the average of cases, this will work no hardship on either party. A better way, however, is to make proper allowance for the preparation of the rock and for the extra wall required, and omit any estimate for paving where no paving is actually placed. In this way, the price paid for each structure will properly represent its reasonable cost, without relying on an over-payment for one structure to offset the underpayment for another. If there are different contractors, *A* will hardly be soothed for his losses by knowing that *B* has made a large profit.

TUNNELS

TUNNEL SURVEYING

TUNNEL ECONOMICS

1. **Importance of Careful and Accurate Work.**—The location and the construction of tunnels are so intimately connected that it is considered best to treat them together instead of making tunnel location a part of railroad location. The cost of constructing tunnels is comparatively so great, and the consequences of a blunder so far reaching, that an error in location is a very serious matter. It is, therefore, of the utmost importance that every phase of the location should be studied with especial care, and that special pains should be taken to perform the surveying work so as to obtain exceptionally accurate results.

2. **Choice Between a Tunnel and an Open Cut.**—An engineer frequently has to decide for himself whether, under certain conditions, a tunnel is economically preferable to a cut. This question is so broad that only a general idea of the method of solution can be given here. There are three general cases in which a tunnel is preferable; namely, (1) when the depth is so great that an open cut is out of the question; (2) when the tunnel is merely a subsurface tunnel, the surface being utilized for other purposes; (3) when the depth is such that it is cheaper to tunnel than to make an open cut.

In the first case, there is no alternative except, possibly, an utter change in the line of the road for a considerable distance. The second case is usually decided by considerations that have little or no relation to engineering. The third case is purely an engineering question. It is generally assumed that, when the depth of cutting exceeds 60 feet, it becomes expedient to drive a tunnel; but, unfortunately, the problem cannot always be solved by so simple a rule. A very soft soil, when dug out for an open cut, requires comparatively flat side slopes and therefore an enormous volume of cut when the center depth is great. A tunnel in such soil will be easy to dig, but will require expensive and strong timbering. An open rock cut may be made with steeper side slopes and much less volume of cutting. A tunnel through such rock may require little or no timbering, but the rock will be difficult to blast out. The requirements for material for fill in the vicinity of the location have considerable influence on the question. If a large amount of material is required to make a near-by fill, it will be justifiable to make an open cut to a depth much greater than that at which a tunnel would otherwise be more economical.

3. Approaches.—The open cuts at the ends of a tunnel are called **tunnel approaches.** The engineer must decide when it is expedient to discontinue open cutting and begin tunneling. The principles regarding relative cost of an open cut and of a tunnel must again be used. Where the slope of the hill that is tunneled is very steep and the line approaches the hill in the direction of the steepest slope, the length of the approaches will probably be very short. On the other hand, when the hill slopes are comparatively flat, there may be a very long open cut before reaching a depth where a tunnel would be either economical or practicable. Even the tendency of the approaches to fill with snow in winter, or to be blocked by landslides, may carry some weight in deciding on the better plan to adopt.

4. Alinement and Grade.—There are many economical reasons, from the standpoint of both construction and

operation, why a tunnel should be built straight. This, however, is sometimes impracticable, except by the adoption of a design that would be too expensive in other respects. When a tunnel forms a link in a long maximum grade of the road, there is the temptation to continue that grade through the tunnel; but it should be remembered that the slipping of the driving wheels on the wet rails in a tunnel, as well as the increased atmospheric resistance, reduces very materially the capacity of a locomotive. The grade in a tunnel should, therefore, be materially reduced below the grades outside. It is often preferable to reduce the grade so as to make it as nearly level as is consistent with proper drainage, say .2 per cent. If the tunnel is very long, it is preferable to have two grades, each rising from the ends toward the center. Where the tunnel must form part of a long grade, the grade may be all in one direction. Ventilation will be somewhat assisted by having the grade all in one direction. In constructing a long tunnel on the Great Northern Railroad, an allowance of .5 per cent. in the grade was made. The tunnel formed part of a grade of 2.2 per cent., and the part in the tunnel was reduced to 1.7 per cent.

LINES AND LEVELS

5. Preliminary Operations.—The decision to construct a tunnel at a given locality implies a previous surface survey of the entire length of the proposed tunnel. When the location of the tunnel has been definitely determined, the next step is a very precise resurvey of the line of the tunnel on the surface. Tunnels are usually constructed by excavating from each end, and sometimes by excavating in each direction from the foot of shafts; this requires the most minute accuracy in all the surveying operations, so that when the headings meet there will be no appreciable discrepancy in the two or more surveyed lines, either laterally or vertically. If the tunnel is very short, the ordinary engineers' transit, when properly handled, will give as close results as are necessary. When the tunnel is very long, it is almost

essential to have a special instrument, called a **tunnel transit,** by which a great degree of accuracy can be obtained. The telescope of a tunnel transit is unusually large and powerful; it can be taken out and inverted in the wyes, which are open, and is fitted with a striding level by which the transverse axis can be made truly horizontal. Whatever instruments are used, they should be tested often and kept in perfect adjustment.

The approximate line of the tunnel is first run with ordinary instruments, by means of which a series of points is determined, each intermediate point being so located that the two points between which it lies are plainly visible from it. In Fig. 1, *e p q n g* represents a section of a hill to be tunneled, and *a*, *b*, *c*, *d* are points determining the line of survey. Intermediate points *h*, *p*, *k*, *q*, etc. are also marked. The

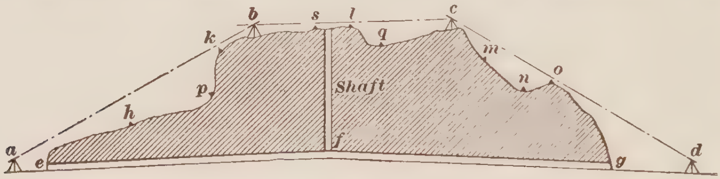


Fig. 1

points *a*, *h*, *p*, *k*, *b*, etc. should be marked by means of large plugs firmly set in the ground; these plugs may be made of 6″ × 6″ timber. By means of repeated observations, in which all errors of adjustment are carefully eliminated, points that are exactly on a line may be determined on the tops of these plugs. The plugs (or "monuments") should be set into the ground so far that they will not be affected by frost. It is wise to fill the pit around these monuments with broken stone or even concrete, well rammed in. The work of the surface survey is preferably done in calm, cloudy weather. The early morning is the best time for this kind of work, provided that the wind is not blowing; the errors due to refraction are then least. Some engineers prefer to do the work at night, using plummet lamps to sight on. The center surface lines of several long tunnels now in operation were run in this manner.

As shown in Fig. 1, points have been established not only at *a*, *b*, *c*, and *d*, but also at several intermediate points. These intermediate points may be utilized when the points at a greater distance are temporarily invisible on account of fog. By setting up a transit at *a* and *d* and obtaining alinement from some one of the previously established points, intermediate alinement points may be rapidly located.

6. Levels.—A line of levels must be run over the surface, determining the elevations of all points. There is nothing peculiar about this leveling, except that, since the slopes are usually very steep, it will require unusual care to avoid error. As far as possible, the foresights and backsights must be made nearly equal, so as to neutralize errors in the adjustments of the level.

7. Determination of Horizontal Distances.—Horizontal distances may be determined in one of two ways.

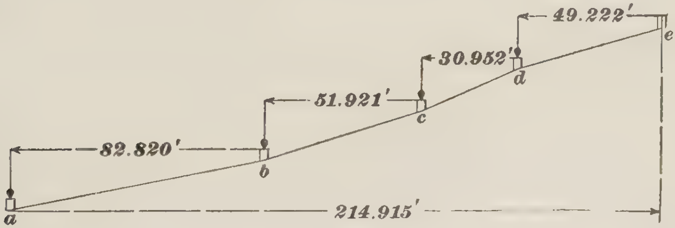


FIG. 2

The usual method of making precise tape measurements, by holding the two ends of the tape constantly level, as illustrated in Fig. 2, may be followed. This method will serve for measuring the line over the comparatively level section that is sometimes found over the middle part of the tunnel (as from *b* to *c*, Fig. 1); but near the ends of the tunnel the slopes are usually so steep that it is impracticable to hold any great length of tape truly horizontal. The use of this method in such cases will require that the plugs shall be placed comparatively close together, and that the position of the end of the tape over each plug shall be plumbed down

with a plumb-bob. While this method is theoretically precise, there is a certainty of a small inaccuracy at each point, and the accumulation of a very large number of these, although some of them will counterbalance, will make the final result somewhat uncertain.

8. For very precise work, the length of the tape must be corrected for temperature. The length of a steel tape increases by about .0000065 of itself for every degree Fahrenheit that the temperature rises, and decreases by an equal proportional amount for every degree Fahrenheit that the temperature drops. Thus, if the tape has a length of 100 feet at 60°, its length at 90° will be

$$100\,[1 + (90 - 60) \times .0000065] = 100.02 \text{ feet},$$

and its length at 25° will be

$$100\,[1 - (60 - 25) \times .0000065] = 99.98 \text{ feet}$$

Another thing that must be taken into account is the force with which the tape is pulled. The cross-sectional area of a fine steel tape may be as small as .002 square inch. Assuming that the modulus of elasticity of the tape is 30,000,000 pounds per square inch, this means that a pull of 15 pounds will stretch the tape $\frac{1}{4000}$ of its length, or .025 foot, which is about .3 inch. Besides, when the tape is swung clear from the ground, as is usually necessary in field operations, the sag of the tape is equivalent to an increase in its length, since the distance between its two ends, as registered, is evidently greater than the true distance. Elaborate calculations are sometimes made to determine the true distance between the ends of the tape under a given tension. Usually, such calculations can be, and are, avoided by determining what is called the **normal tension** on the tape. The normal tension is such a tension that the increase in length of the tape due to its stretch is exactly compensated by the loss in horizontal distance due to the sag. This normal tension can be obtained without any calculations by stretching the tape on a level floor with just enough tension to make it taut, marking its length on the floor, and then raising the tape clear from the floor and noting by means of a spring balance

and plumb-bobs just what tension is required to make the horizontal distance between the ends the same as before. In making this test, the greatest care must be taken that the ends of the tape are perfectly steady, and that the plumb-bobs are used to project vertically the exact position of the end marks downwards to the floor.*

9. The second method of determining horizontal distances, and the only one that can be conveniently used on very steep slopes, is illustrated in Fig. 3. The inclined distances *a b*, *b c*, etc. are measured directly with the tape, and then the differences of elevation, or vertical distances *a a′*, *b b′*,

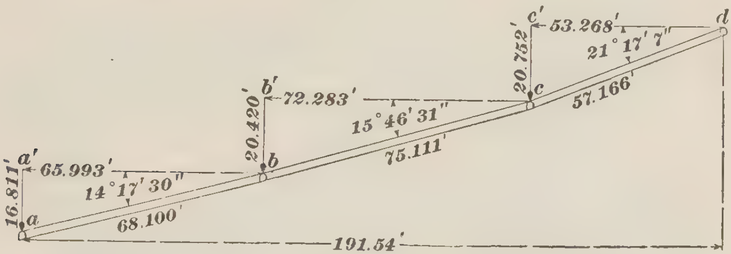


FIG. 3

etc., are determined from the level notes. The true horizontal distance may be determined for each section: (1) by subtracting the square of the vertical distance from the square of the inclined distance, and extracting the square root of the difference; (2) by the methods employed in trigonometry for the solution of right triangles.

EXAMPLE.—The slope distance *a b* was found to measure 68.10 feet; the difference of elevation between *a* and *b* was found from the level notes to be 16.811. What was the horizontal distance *a′ b*?

SOLUTION.—The right triangle *a b a′* gives

$$a' b = \sqrt{a\,b^2 - a\,a'^2} = \sqrt{68.10^2 - 16.811^2} = 65.993. \quad \text{Ans.}$$

Otherwise, thus:

$$\sin a\,b\,a' = \frac{16.811}{68.10}; \quad a\,b\,a' = 14°\ 17'\ 30''$$

$$a' b = 16.811 \div \tan 14°\ 17'\ 30'' = 65.993. \quad \text{Ans.}$$

*Further details relating to temperature and tension corrections are given in *City Surveying*.

EXAMPLES FOR PRACTICE

1. Assuming the inclined and vertical distances given in Fig. 3, verify the values for $b'c$ and $c'd$.

2. The inclined distance between two plugs is 29.746 feet. The elevations of the two plugs above datum are, respectively, 196.349 and 213.073 feet. Find the horizontal distance. Ans. 24.599 ft.

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10. Curved Tunnels.—Curved tunnels should be avoided where practicable; but, as may readily be seen

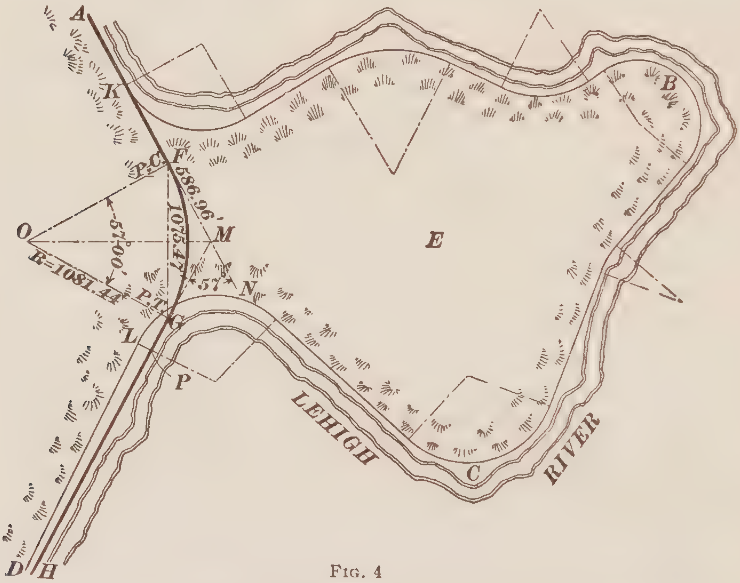

FIG. 4

by the case illustrated in Figs. 4 and 5, it is sometimes impossible to avoid them. In such a case, the tangent distances are calculated and the P. C. and P. T. are located on the surface by direct measurement. The work and calculations are repeated many times, and every possible precaution is taken to secure perfect accuracy of results. The sketch given in Fig. 4 shows the difficulties attending the laying out of the Rockport tunnel on the Lehigh Valley

Railroad. The original line $ABCD$ followed the course of the Lehigh River, which hugs the bluff E. The tunnel line $AFGH$ would have been adopted, and the tunnel driven when the road was first constructed, but a rival line was building on the opposite side of the river, and there was a race to reach the Wyoming Valley coal fields and command the coal traffic. The tunnel line was accordingly postponed, and the river line adopted. The tunnel was driven in 1882–83, after a lapse of 20 years. The neck FG through which the tunnel passes (a profile of which is shown in Fig. 5) reached a height of more than 300 feet. The hill-sides were so steep that in places a man could hardly stand. The grade of the original line $ABCD$ is about 20 feet per mile, rising from A to D, and the length is $1\frac{1}{2}$ miles greater than that of the tunnel line $AKFGP$. The grade on the new line from K to P was made 20 feet per mile, the same

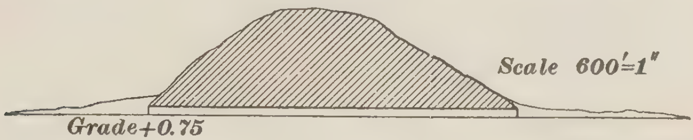

Fig. 5

as the grade of the tangent AK of the old line; this brought the point P 30 feet lower than the point L. The new line PH was then given a grade of 40 feet per mile so as to gain on the old line LD at the rate of 20 feet per mile, coming to the same elevation $1\frac{1}{2}$ miles from L and P. The new tangent PH was then located so as to meet LD at a distance of $1\frac{1}{2}$ miles from L and P. The intersection angle GMN measured 57°, and a 5° 18′ curve was decided on; the tangent distances FM and MG were then measured directly on the surface, and the P. C. and P. T. were set.

On account of the steepness of the slopes and the height of the hill, much difficulty was experienced in making a satisfactory intersection to locate the point M. Within a distance of 500 feet, there was a difference in elevation of more than 300 feet, and some of the sights contained a vertical angle of more than 60°. The lines were run

principally in the early morning hours, though some of the best results were obtained on cloudy days. A large tunnel transit with powerful lenses, and of more than double the weight of an ordinary transit, was used. Common pins against a dark background were used for backsights. First, an intersection was made, large plugs or hubs 6 inches square being used. The tangent KM, Fig. 4, was then repeatedly run, and each line marked on the hubs Q and S, Fig. 6, the hubs being so located that the intersection M was between them. Each line was marked with tacks, each of which was numbered, as shown in the figure. The lines varied each time, no two coinciding. One or two fell wide of the mark, and were ignored. Finally, the mean of the lines was adopted. The tangent GM, Fig. 4, was then run

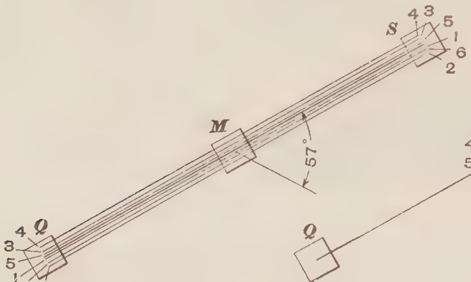

FIG. 6

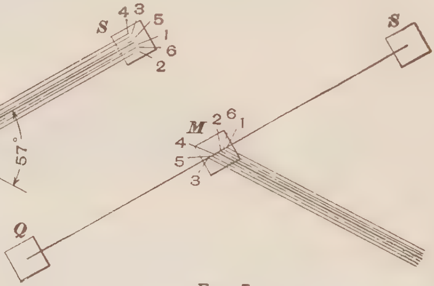

FIG. 7

an equal number of times, and each intersection on the line QS, Fig. 7, marked on the hub M with a tack and numbered. The mean of these intersections was taken as final.

Equally great difficulty was experienced in locating the P. C. and P. T. The distance was measured many times, and each distance marked. The mean was then taken as the correct measurement. The top of the hill had the form of a plateau, and the center O, Fig. 4, of the curve was located by turning a right angle to the tangent KF at F, the P. C., and measuring the radius, 1,081.44 feet. The central angle FOG of 57° was then turned, and the second radius OG run out and measured. The line and measurement falling on the plug at the P. T. at G proved the work correct. The

result of all this care and pains was that, when the tunnel was driven from both ends and the headings met, there was less than $\frac{1}{2}$ inch discrepancy in the two lines.

11. Interior Surveys.—During construction, the alinement and levels for the interior of the tunnel must be located, relocated, and tested with great frequency. Surveying points in the interior of the tunnel must be placed where they are not likely to be disturbed or covered. In general, the points that are least likely to be disturbed are in the roof of the tunnel. Points are usually located by drilling a hole into the roof and filling it with a plug of wood driven in very tight. Into this plug is driven a screw eye, or else a **spud,** which is a nail having a round flattened head with a hole through its center (see Fig. 8). Sometimes, the screw eyes or spuds are driven in the timbering of the tunnel; but, as the timbering is likely to be jarred out of place, such locations should be tested very carefully and as frequently as possible, especially when the line is being carried forwards.

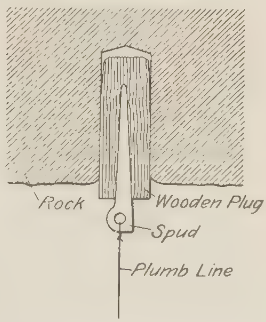

FIG. 8

A transit can be centered under a roof point by setting the instrument in such a position that the center of the telescope will be directly under the point of a plummet suspended from the roof point. A transit telescope usually has a small hole drilled in the top of the ball that forms the connection between the telescope and the axis. When the telescope is horizontal, this hole is exactly over the center of the instrument, and the transit can easily be placed so that the hole is directly under the plumb-bob point. Another method is to determine, by means of a plummet, a temporary point on the floor of the tunnel directly under the spud, and then center the transit over that point in the usual manner. This method requires two operations instead of one, and usually is not so good as the other.

12. In underground work, it is always necessary to illuminate the cross-wires of the transit or level. This may be done by holding a bull's-eye lantern so that its light will shine into the telescope and yet be outside of the direct line of vision. A preferable method is to use a reflector, such as is illustrated in Fig. 9. In this case, the lantern may readily be held by the transitman (or preferably by an assistant) in such a position that its light will be reflected from the reflector directly back on to the cross-wires, but without interfering with the line of sight. For leveling work, the face of the level rod must be illuminated sufficiently to determine the readings. A transit pole or a plumb-line may similarly be illuminated as a target for the transit work. A preferable method is to use a plummet lamp, such as is illustrated in Fig. 10. This lamp has an oil reservoir of brass, with a steel point at its lower end. A cylindrical stem contains a wick at the upper end. The lamp is suspended by gimbal points to a bail, to which a wire is attached. This wire can be run through the eye of a spud or screw eye, and the plumb will then be vertically under the spud. To locate a point on a given line by means of such a plummet lamp the lamp is suspended by the wire and shifted laterally until the line of sight bisects the flame. If, at the same time, the upper end of the wire is held against the roof of the tunnel, the point in the roof that is in the desired line is readily determined and may be marked with a plug, as previously described.

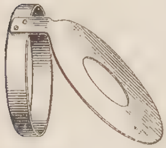
FIG. 9

FIG. 10

Grade points, also, may be located in the tunnel by means of these plugs in the roof. It is only necessary to invert the level rod so that its zero end is against the point in the roof. The rod reading then indicates the distance of the cross-wires below the point in the roof. To determine the elevation of the plug, the rod reading must

be added to the height of the instrument, instead of subtracted, as in ordinary level work. Roof plugs may be used also as bench marks, but bench marks are sometimes made by drilling a hole horizontally into the side of the tunnel at some convenient height, into which plugs of wood or iron are firmly driven. These plugs project far enough from the wall to allow a level rod to be held on them in a vertical position. In a timber-lined tunnel, large nails or spikes may be driven horizontally into the wall posts and serve the same purpose, although there is some danger that the timbers may shift their position sufficiently to destroy the accuracy of the work. These points should, therefore, be frequently tested.

13. If the tunnel is curved, it is necessary to determine the true center line at very frequent intervals, in order that the cross-section of the tunnel may be sufficiently large at each point, and yet not excessively or wastefully so. The location of the points of the curve is effected by the methods explained in *Circular Curves*.

It should be noted that, since it is easier in tunnel work to locate points on a tangent and then make some offsets, it is better to locate a number of lines that are all tangent to the curve and that have their intersections mutually intervisible, which of course means that they all lie within the cross-section of the tunnel. The lengths of these tangents and the angles between them can be easily computed. The desired points of the circular curves may be located by offsets from these tangents.

14. Surveying Down a Shaft.—In order that the excavation at the bottom of a shaft may be properly directed, it is necessary to transfer from the surface to the bottom of the shaft the three surveying elements of *distance*, *elevation*, and *direction*. Since the shafts are almost invariably made vertical, it is possible to suspend a heavy plumb-bob by means of a wire of sufficient length from the top to the bottom of the shaft. The element of distance, which in this case means the horizontal distance of any given point from a reference point, is very readily transferred, with sufficient

accuracy, from the top to the bottom of the shaft by means of the plumb-bob. Similarly, the difference of elevation may be measured with all necessary accuracy by merely measuring vertically down the shaft with a steel tape. The third element, that of direction, requires great care, and often taxes the skill and ingenuity of the locating engineer. In its bare

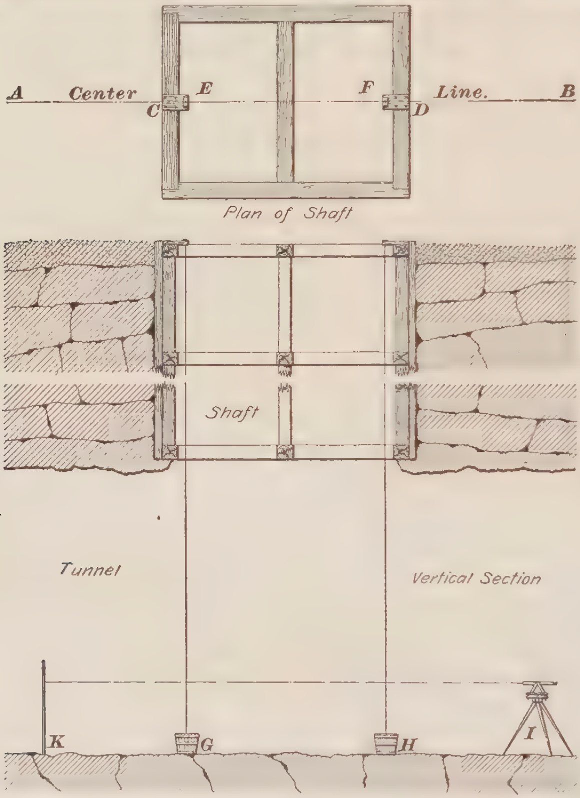

FIG. 11

outline, the method is simple and perhaps looks easy. Practically, the chances for inaccuracy are large and precision is very difficult. The details of the method generally used are as follows: Two pieces of plank C and D, Fig. 11, are spiked to the shaft timbers where the center line of the surface survey crosses; the edges of the plank project inwards

over the shaft wall. Slots are cut in the edges E and F of the planks C and D on the center line. An iron plate with a carefully drilled hole in its center is placed over each slot and in the center line of the surface survey. Holes are drilled in the corners of the plates, which are fastened to the plank. Plumb-bobs weighing from 20 to 30 pounds are suspended by fine steel wires that pass through the holes in the plates. In order to prevent the vibration of these wires, each plumb-bob is entirely immersed in a pail of oil placed at the bottom of the shaft. When the plumb-bobs come to rest, the lines that suspend them are exactly in the center line, as laid down on the surface of the ground. The tunnel having been driven some distance from the foot of the shaft, a transit is set up in it at some point I as far as possible from the wires that carry the bobs at G and H, and moved until its line of sight coincides with the line determined by the two wires—that is, until the vertical wire of the transit covers both of the wires that carry the bobs. A plug is set on line at some point K, the instrument is moved to K, and a back-sight taken on I, the latter point having been previously marked by a plug; the line from K to I should intersect both supporting wires. As the points E and F are usually not more than about 8 feet apart, and as the heading may run several thousand feet from the shaft before meeting the heading from the other direction, a very slight error in the location of the points E and F will be greatly magnified. It is also found that drafts of air, which are invariably found in a shaft, prevent the wires from hanging truly vertical, and hence the horizontal line passing through the wires at the bottom of the shaft may or may not be parallel to the surface line. When the heading from a shaft meets the heading from another shaft, or from one end of the tunnel, the alinement should be verified and corrected as much as necessary, even though the correction requires a slight change in alinement.

TUNNEL CONSTRUCTION

CROSS-SECTIONS

15. Single-Track Tunnels.—The usual general form of cross-section for a railroad tunnel consists of an arched roof with vertical or nearly vertical side walls. The details, however, vary according to the character of the material excavated. The dimensions of a tunnel section vary according to the form of section and the number of tracks in the tunnel. For a single-track tunnel, the following dimensions are considered suitable: in hard rock with no lining, from 14 to 16 feet wide at the bottom; 16 feet wide at from 14 to 16 feet above the bottom; and the roof arch on a curve of 8 feet radius. The same general dimensions are suitable for a single-track tunnel lined with timber or masonry in the ordinary manner; in this case, the tunnel excavation is made sufficiently large to allow for the thickness of the lining outside the regular tunnel section. In Fig. 12 is shown a good form of unlined tunnel section, such as is adopted for solid unstratified rock,

Fig. 12

where there is no danger of slips or falling rock. In **Fig. 13** is shown a good form of section where the tunnel is lined with concrete.

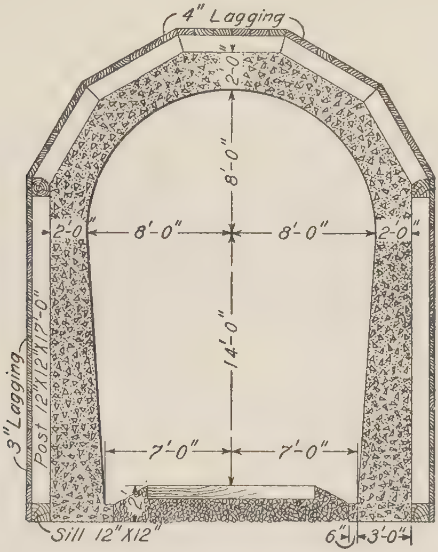

Fig. 13

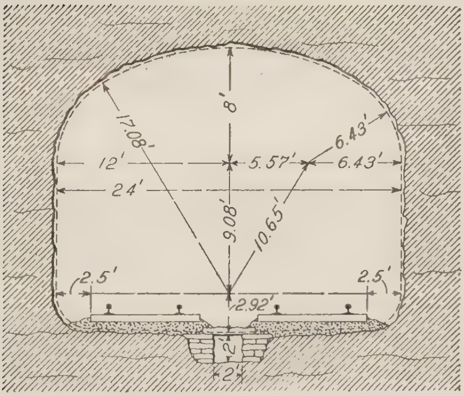

Fig. 14

When a tunnel is built on a curve, it is customary to make the section a little wider than that for a straight tunnel.

16. Double-Track Tunnels.—For a double-track tunnel, the form of section shown in Fig. 14 may be used. In some cases, instead of a three-centered arch, as shown in the figure, the roof arch consists of a semicircular curve of $13\frac{1}{2}$ feet radius.

17. Special Forms of Tunnel Sections.—In some kinds of material, it is necessary to use special forms of

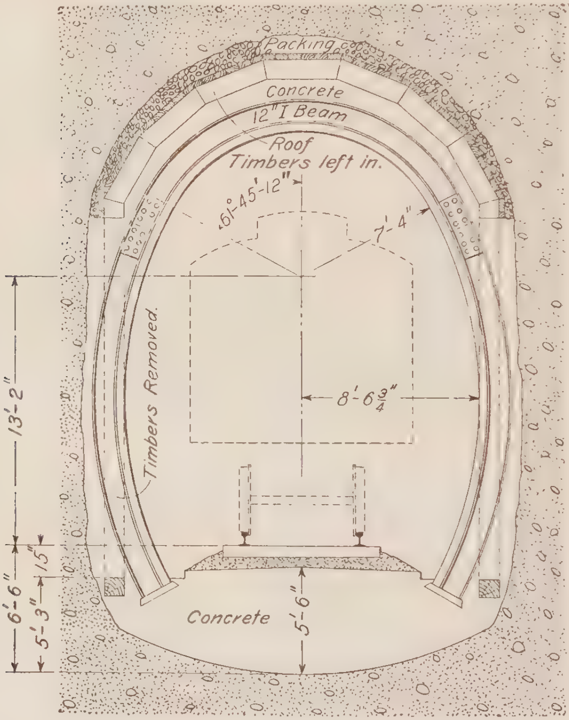

Fig. 15

sections in order to permit a suitable form of lining to resist the pressure on the roof and sides of the tunnel. In Fig. 15 is shown a form of section used in a portion of the Aspen tunnel on the Northern Pacific Railroad. The interior dotted outline represents the cross-section of a passenger car and shows the amount of clearance between the sides of the car and the walls of the tunnel.

METHOD OF CONSTRUCTION

18. Headings.—The almost invariable method of driving a tunnel is to excavate first a **heading** (see Fig. 27) whose cross-sectional area is about one-fifth or one-fourth of the total cross-section of the tunnel. The rest of the cross-section is called the **bench.** Frequently, the heading is driven through the upper part of the cross-section, as this renders it somewhat easier to place the timbering during enlargement. If the ground is very wet, there is an advantage in making the heading near the bottom of the section,

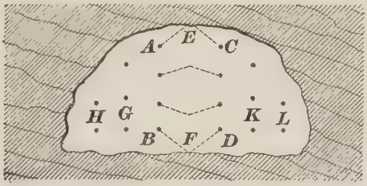

FIG. 16

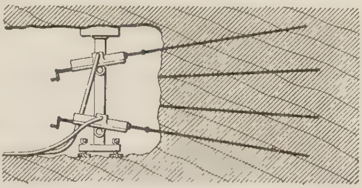
FIG. 18

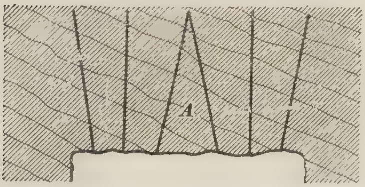

FIG. 17

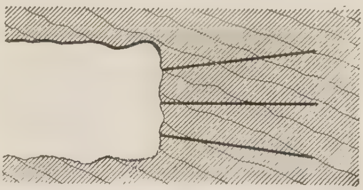
FIG. 19

as then the bottom of the heading will act as a drain, and the soil will be much drier and firmer by the time the enlargement is commenced. The heading is usually made about 5 feet wide and from 6 to 8 feet in height. The face of the heading is usually kept at least 50 feet in advance of the bench (see B and B', Figs. 26 and 27). Sometimes, two headings are driven along the lines that will ultimately be the two lower corners of the tunnel section.

When the material encountered is a hard self-sustaining rock, the work of excavation is comparatively simple, and, on account of the fact that there is no necessity of timbering, the work is actually less expensive than for very soft

material, which must be heavily timbered. This work is usually done with power drills worked with compressed air. The plant for operating these drills will be considered in a subsequent article. Drills working in headings are usually mounted on columns, two drills on each column (see Fig. 18). The drills working on the bench are usually mounted on tripods (see Fig. 26). The air pipe is carried to within about 50 feet of the bench, where a bench hose of equal diameter is attached to it. At the end of the hose is a metal nozzle, called a **manifold,** containing hose connections for each of the drills.

A section of heading showing the arrangement of drill holes in the face is given in Fig. 16. The two middle rows

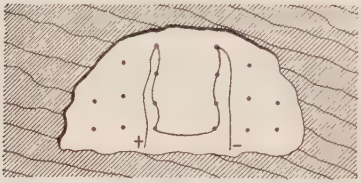

FIG. 20

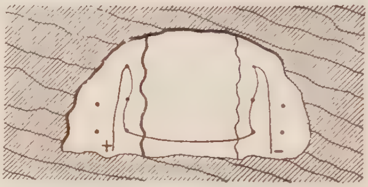

FIG. 21

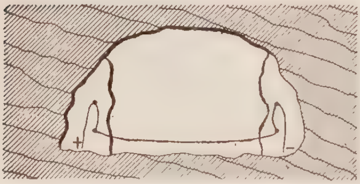

FIG. 22

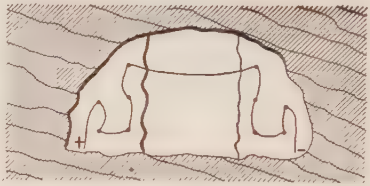

FIG. 23

of holes AB and CD converge at an angle of about 20°, meeting nearly on the centre line EF of the tunnel; they are called the **center-cut holes.** The mass of rock included by these holes is wedge-shaped, as shown by the plan in Fig. 17. The removing of this wedge by blasting is called **breaking the cut.** Fig. 18 shows a longitudinal section through the center-cut holes. The rows of holes G, H, K, and L, Fig. 16, on each side of the center-cut holes are called **side rounds.** If there is but one row on each side, the rows are called **single side rounds;** if two rows, **double side**

rounds. A longitudinal section through the side holes is given in Fig. 19. The cut and the side rounds are loaded at the same time. The cut is fired first (see Fig. 20). The side rounds are either fired separately—that is, one row on each side of the cut at a time (see Figs. 21 and 22)—or they are double-fired—that is, both rows simultaneously, as shown in Fig. 23. In Figs. 20 to 23 the irregular lines that connect the holes represent wires through which electric currents are passed to ignite the blast.

19. Enlarging the Heading.—In that portion of the heading shown in the figures just mentioned, the holes are

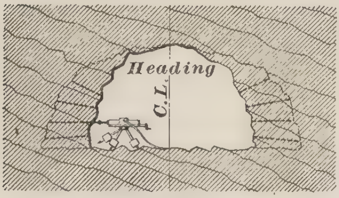

FIG. 24

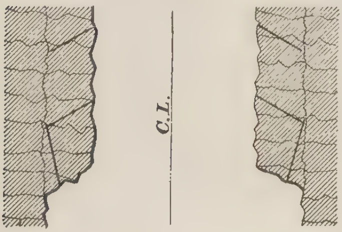

FIG. 25

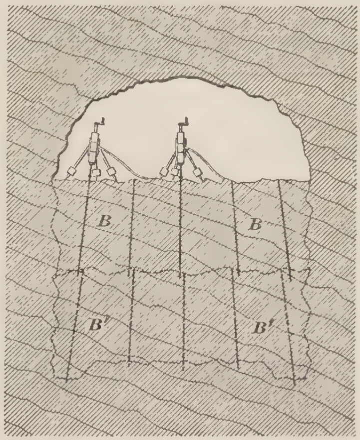

FIG. 26

drilled directly into the face of the heading. After they are fired and the material is removed, side holes are drilled at an angle of about 60° with the center line, denoted by the letters CL, as shown in section in Fig. 24 and in plan in Fig. 25.

20. Removing the Bench.—The bench is taken out in two sections, B and B', as shown in section in Fig. 26. The full tunnel section is shown by dotted lines.

The holes in the bench are inclined backwards from a vertical line. A longitudinal section through the center line,

showing the usual mode of drilling headings and benches, is given in Fig. 27. The center-cut holes in the heading *H* and all the bench holes at *B* and *B′* are usually fired together, and then the double side rounds in the heading. The center cut offers the greatest resistance to blasting. The holes are consequently loaded with more powerful explosives than are used for either side rounds or bench.

In driving the New York aqueduct tunnel, the cut was loaded with dynamite containing from 60 to 80 per cent. of nitroglycerine, while the average bench powder contained but 40 per cent. of nitroglycerine. On some sections, where

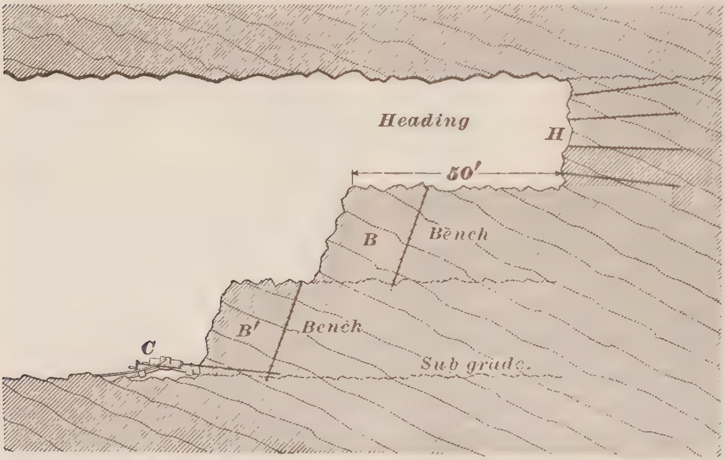

Fig. 27

rock of special hardness was encountered, the cut was loaded with pure nitroglycerine. This operation, however, is always attended with great danger. After several premature explosions, resulting in considerable loss of life, the use of pure nitroglycerine was abandoned. The effect of firing the cut is generally to pulverize the rock, and all tunnel blasting is intended so to break the rock as to render the use of sledges unnecessary in reducing masses of rock to sizes convenient for loading.

The efficiency of the powder depends largely on the judgment used in locating the holes and on the angle at which

they are bored. The position of the machine while drilling holes at the foot of the bench is shown at *C*, Fig. 27.

21. Power Plant.—The plant for furnishing the power used in driving a tunnel will vary in size according to the length of the tunnel. For a tunnel of moderate length, a single plant located near one of the portals will usually provide all the power necessary to run the machinery for drilling and pumping that is used in both headings, and also to ventilate the tunnel. For a tunnel of considerable length, it is usually better to install a power plant at or near each end. In short tunnels through rock, it is sometimes the practice to work the power drills by steam without the use of compressed air; in such cases, the steam is furnished from the boiler direct to the drill. In driving the Pryor Gap tunnel, on the Burlington & Missouri River Railroad, three power drills were used in each end, the drills taking the steam through hose from the boiler. A 16-horsepower upright boiler furnished sufficient steam to run the three drills. In one of the tunnels, previously referred to, fourteen power drills worked by compressed air were used at each end of the tunnel. All material taken into or out of the tunnel during construction was hauled by electric motors; the tunnel was lighted by electricity, and the foul air was drawn out by means of large centrifugal fans. In addition, a large quantity of water was pumped through the tunnel from the east end, which was at the upper end of the gradient. In order to furnish sufficient power to run all this machinery, a large power plant was installed at each portal of the tunnel. At the east portal, six 150-horsepower boilers with air compressors, pumps, dynamos, and fans were required; and at the west portal, three 150-horsepower boilers with the necessary machinery were used.

22. Ventilation.—When the tunnel heading is not very far from the mouth of the tunnel, and especially in clear weather, the gases formed by the combustion of the powder used in blasting will clear away with sufficient rapidity not to hinder the work; but in dull, foggy weather, and when the

distance from the heading to the mouth of the tunnel is great, several hours may elapse after a blast before the tunnel heading will become even reasonably free from smoke. Contractors frequently make the serious mistake, from a false idea of economy, of neglecting to take the means necessary to clear the air so that men can properly work in it. Men cannot and will not do full work in a tunnel reeking with powder smoke. To do effective work, each man should have a supply of 100 cubic feet of pure air per minute. If there are twenty-five men in the heading, the supply of pure air should therefore be 2,500 cubic feet per minute. The 70-horsepower compressor that will be necessary to run the drill in such a heading will deliver to the drills about 500 cubic feet of free air per minute. So far as it goes, this is available for ventilation, but it only supplies one-fifth of the amount of air that is really necessary. The remainder may be supplied by blowers; but this method is objectionable on account of the fact that, when the pure air is forced into the tunnel by blowers, this means that the foul air must be displaced along the total length of the tunnel to the outlet. A preferable method is to use exhaust fans, which remove the powder smoke and foul air through pipes directly from the heading, and which by suction draw in a supply of fresh air from the outside of the tunnel.

23. Lighting.—The older method of lighting a tunnel during construction was by the employment of miners' lamps or of candles. These are exceedingly objectionable, because they not only consume the supply of fresh air, which in a tunnel has a considerable value, but they also add still more to the smoke of the tunnel and give out an offensive smell. Such a method of lighting has been almost entirely superseded by the use of electric lights, provided that a power plant is used in connection with the construction of the tunnel. Electricity has the advantage of furnishing a brilliant light, which consumes no oxygen and makes no smell or smoke. Almost its only disadvantage lies in the

fragility of the globes and the danger of their being broken during the blasting.

24. Shafts.—A **tunnel shaft** is an opening or passageway extending from the surface of the ground to the grade of the tunnel. Shafts are usually made vertical and on the center line of the tunnel. They are used during construction to facilitate the work of excavating by affording access to the tunnel at intermediate points between the portals. Work is carried on in the tunnel in opposite directions from the bottom of a shaft; this shaft serves as a tunnel entrance through which excavated material is hoisted and surplus water pumped; and also as a passageway for pipes conveying compressed air and for admitting necessary construction material. Shafts are also useful in affording ventilation to adjacent sections of a tunnel and in facilitating ventilation through the tunnel after the excavation reaches the shaft from the portal.

25. Shaft Lining.—When a shaft is sunk through solid rock, the walls are self-sustaining and usually require no lining except a curb at the top of the shaft. When, however, a shaft is sunk through loose material, it is necessary to use a **lining** in order to prevent the sides from caving in. Such a lining is usually made of timber, and is put in, as the shaft is sunk, in the following manner: Rectangular wooden frames, shown in plan at *a*, Fig. 28, are made of the required dimensions to fit inside the shaft. These frames are spaced about 4 feet apart vertically as the shaft is sunk, and behind them is placed **lagging** on end, and in close contact, as shown at *b*. Such lagging may consist of sawed timber or of half-round poles split from young trees, as may be more convenient. As each frame is placed in position, it is supported by struts footing on the bottom of the shaft; or, if the walls are sufficiently firm, the frames are held in place by wedges, until another set is required, when timber struts *c*, mortised into the frames, form the permanent support. These struts are placed one above the other, and, together with the frames into which they are mortised, form continuous timber

columns extending from the bottom to the top of the shaft. With each set of timbers, a horizontal timber *d*, called a **bunton**, is placed with ends abutting against the vertical timber *e*. A beveled seat with a square shoulder is cut on the vertical timber for each bunton. The buntons are held in place by wedges shown at *f*; these wedges are forced between the bunton and the shoulder of the beveled seat.

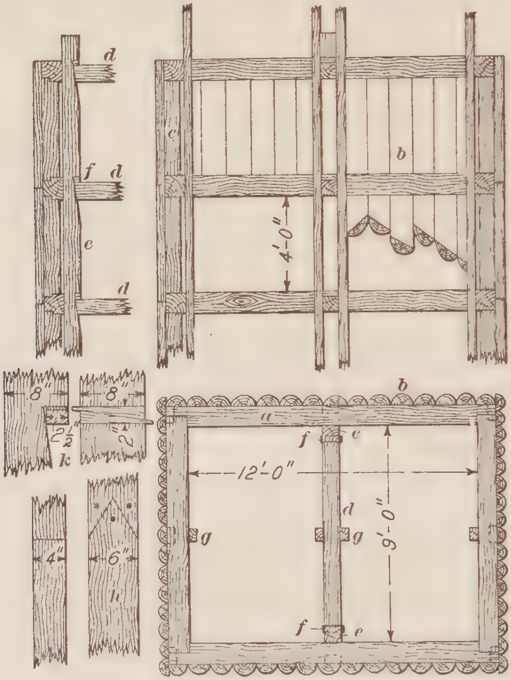

FIG. 28

As the wedges are tightened, the bunton is forced downwards until it is perfectly rigid. Vertical timbers *g*, *g* are spiked to the buntons and to the ends of the frames, to serve as guides for the carriage. A detail of the splice of the carriage guide is shown at *h*, and of the wedges at *k*. Since the pressure of the earth against the sides of the shaft lining is usually sufficient to hold the timbers in place, there will not be much necessity for using spikes or bolts for such

purpose. Where fastenings of some character are required, it is better to use wooden pins in preference to iron, since most shafts are wet and iron will be liable to rust rapidly.

26. Removing Excavated Material.—The material excavated in tunnel driving is called **muck.** Various methods are employed for handling the muck, which is removed by loading it into dump cars at the foot of the bench and hauling it away. When there is a sufficient descending grade from the bench, the loaded cars can be run by gravity either to the foot of a shaft, where the material is hoisted to the surface, or to the waste dump outside the tunnel entrance. Where the grade will not permit running the cars by gravity, they are hauled by mules or by some form of steam or electric motor. The muck cars are operated on a track of 2 feet or 3 feet gauge, and are of capacities varying from 1 cubic yard to 3 cubic yards each. Usually, a double track is built, which allows loaded and empty cars to pass on separate tracks without delay. When the cars are hauled by mules, a single loaded car usually constitutes a load; but when a motor is used, the cars can be moved in trains of from ten to sixteen cars. When a single track is used, it is generally laid on the center line of the tunnel, with passing branches at suitable intervals.

In Fig. 29 is illustrated a good arrangement for handling the muck from a bench and heading over a single track, the same arrangement being suitable for a double track. At a distance of about 100 feet from the bench, a simple switch is built and two tracks C and D are laid to the bench, as shown. A plank runway F extends from the heading out over the tracks, being supported at suitable intervals on scaffolds E, E. The muck from the heading is loaded into barrows, wheeled on the runway a sufficient distance, and dumped directly into the cars. A simple and effective form of scaffold consists of two pieces of wrought-iron pipe, one telescoping within the other, both being provided with clamps by which they are adjusted to the required length. Such a scaffold is placed in position with the ends of the

pipe abutting against the sides of the tunnel with sufficient force to support the runway and the load on it. The air for working the drills is carried from the air pipe *G* to the bench by means of the bench hose *H*. The manifold *K* attached to the end of the bench hose contains hose connections for

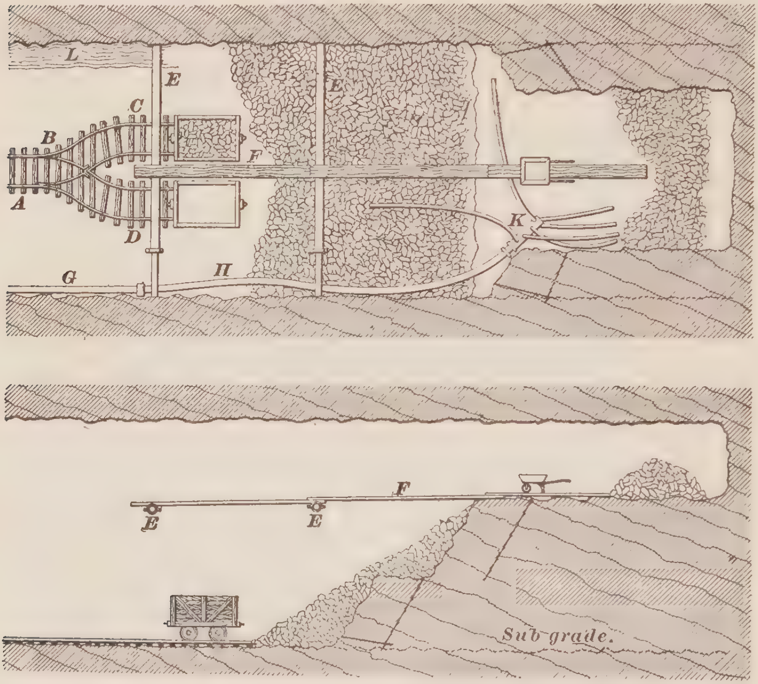

Fig. 29

all the drills. A ditch *L* on the opposite side of the tunnel from the air pipe drains the tunnel.

27. Care of Track.—It is important to keep up a good track on which to haul the cars through the tunnel; and, in order to avoid frequent breakdowns and derailments, the track should be built of heavy material in a substantial manner. As the work advances from the tunnel entrance or from the foot of the shaft, the track will have to be lengthened to cover the increased distance. A common

fault with contractors is a mistaken notion of economy in this particular. Worn-out and cracked rails, poor fastenings, poor ties or very few of them, although apparently cheap, are really expensive, since derailments are continually occurring, which involve costly delays. Short rails of varying lengths are required, so that the track may be kept continually within a short distance of the bench. A supply of rails and ties, together with the necessary track tools, should be constantly on hand, so that the track may be promptly added to as desired. When a single track is used, passing branches are built at suitable intervals, where returning empty cars are switched to pass the loaded cars going out.

FIG. 30

A good form of passing branch is shown in Fig. 30; this is connected with the main track by two switches, which are made self-acting by the simple device shown in the figure. The points of the switch rails *a* and *b* are connected by a clamp rod, attached to a spring *c d*, which is constantly acting and holds the point *a* close against the main rail *e f*, and the switch is constantly set for the passing branch *k l*. The switch points *m* and *n* of the second switch are kept in place by the spring *o p*, the switch being always set for the main track *q r*. An outgoing car running in the direction *r q* finds the switch *Y* set for the main track. On reaching the switch *X*, the flange of the right-hand wheel, passing between the rail *e f* and the switch

point *a*, forces the switch point *b* against the rail *s t*, and the car passes the switch in safety. A returning empty car finds the switch *X* set for the passing branch *k l*; in going from the branch to the main track, the flange of the head-wheel, as it passes between the rail *u t* and the switch point *m*, forces the switch point *n* against the main rail *e f*, and the car passes safely on to the main track. The springs *c d* and *o p* are elastic young saplings kept in place by strong staples driven into the switch ties.

28. Directing the Excavation.—It requires constant attention and watchfulness on the part of the engineers to direct the excavation in its proper direction and elevation. There is a constant tendency to keep above grade, which is possibly due to the unconscious effort to avoid the water that is constantly accumulating. Grade points should be established at least every 25 feet. Since the blasting cannot loosen the rock so that the excavated section will conform closely to the theoretical lines, the points of rock that project into the theoretical section after the blasted material is removed must be taken out by picks, wedges, and sledges. As this is slow and expensive work, it is usually a matter of economy to drill the blast holes so that they will reach a depth of about 1 foot beyond the limiting or theoretical section, so that the entire section will be removed by the blasting.

29. Timbering.—When the material is earth or rotten rock, the tunnel must be timbered. The timbered section should be enough larger than the standard section to admit of a masonry lining. When the material is such that the side walls will stand of themselves for a time, benches *A* and *B*, Fig. 31, are excavated near the springing line, and sets of timbers are placed as shown in the figure. Iron clamps, shown at *C* and *D*, hold the timbers together while the lagging *E*, *F* is being placed. The spaces *G*, *H* between the lagging and the roof are filled with dry rubble or cordwood.

Roof timbers should be either 12 in. × 12 in. or 12 in. × 14 in.; they should be placed with 2 to 4 feet clear space

between two sets. Where the ground is very soft, with a tendency to expand, larger timbers may be necessary. Hemlock, yellow pine, or spruce is commonly used. In special cases, where great pressure is to be resisted, oak is used. When the side walls will not support the roof timbers, the latter are carried on supports arranged as shown in Fig. 32. Four posts *A*, *C*, *D*, and *B*, resting on sills *O*, *P*, *Q*, and *R*, are mortised into the cap *EF*. The roof timbers

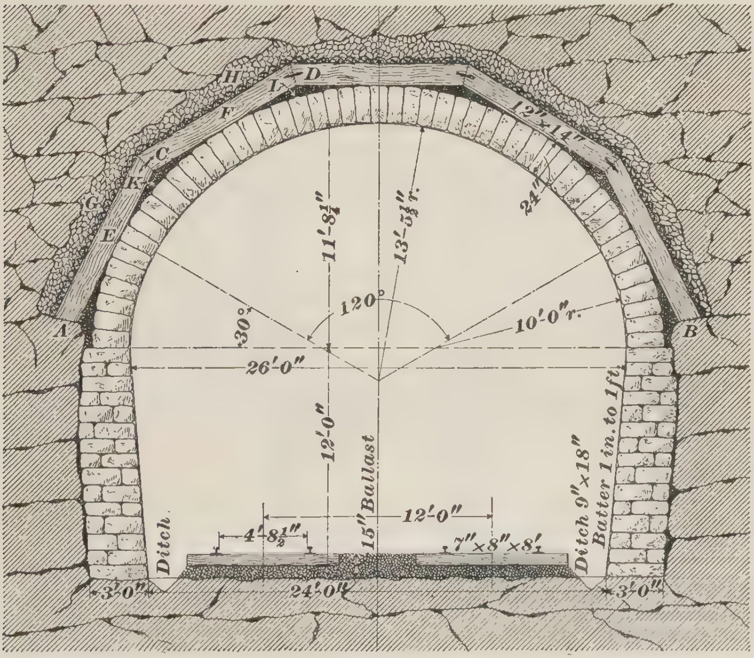

Fig. 31

G, *H*, *K* are clamped together as in Fig. 31, and mortised into the cap at the springing line. The pressure against the roof timbers is relieved by the struts *L*, *M*, *N*, *U*, and *V*, which transfer the pressure to the posts *C* and *D* and the cap *EF*. The dimensions of timber given in the drawing are such as are used where the pressure is great; they will meet the requirements of most situations. The lagging

may be either sawed timber or split poles, obtained by splitting in half straight-grained chestnut or oak saplings. The backing may be either dry rubble or concrete. The side walls are all of well-scabbled rubble of good-sized stones, with even beds, laid in courses with cement mortar. The impost courses S and T should be of well-cut stone not less than 12 inches in thickness and the full width of the wall. The arch is of either brick or rubble. The caps and

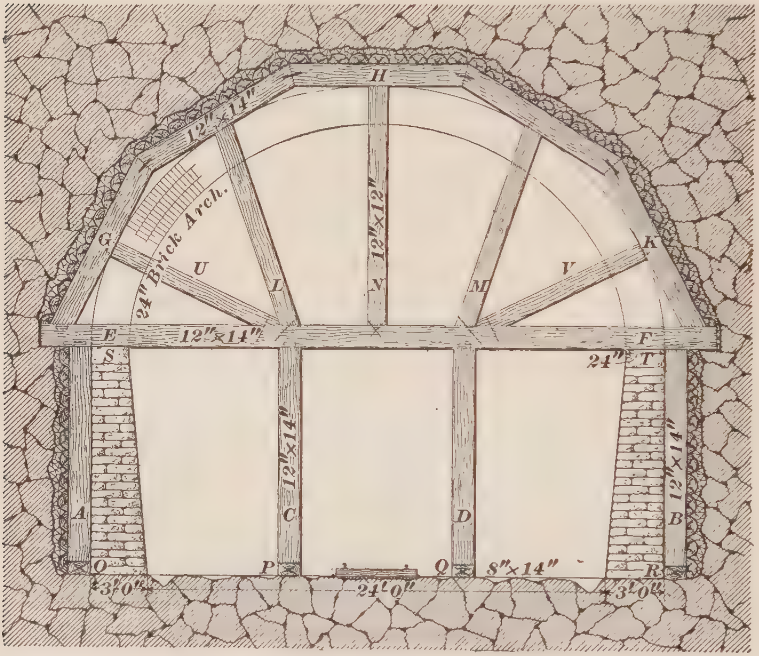

Fig. 32

roof struts interfere somewhat with arching. Holes are left in the masonry where these timbers interfere until a section of the arch is completed, when they are removed and the gaps are filled with masonry, the joints being thoroughly grouted. All other timbers are left in place. The spaces K, L, etc., Fig. 31, between the arch and roof timbers are usually filled with concrete.

30. When the material through which the tunnel passes is very soft, with slight coherence, all the energy and skill of engineer and workmen are required to make headway. It is considered the better practice to drive the heading at the bottom of the tunnel instead of the top, the reason being that, by the time the heading is driven, the ground composing the remainder of the section will have become thoroughly drained, and may be taken out with much greater safety and less expense than would be the case with a top heading. The mode of driving a heading through such material is illustrated in Fig. 33, which represents a cross-section and a longitudinal section of the heading, with a complete system of timbering.

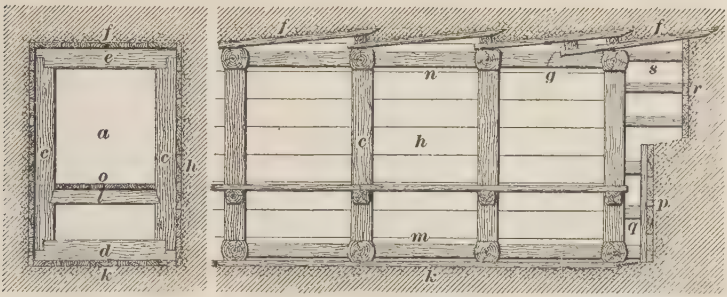

FIG. 33

A full section of timbers, as *c e c d* shown at *a*, is called a **set**; the upright timber *c* is called the **leg**; the horizontal timber *d*, the **sill**; and *e*, the **cap** or **collar**. The short boards *f*, *f* that extend from collar to collar and are in direct contact with the sustained material are called **poling boards**. They are sharpened to a cutting edge, and are driven into the face of the heading with sledges, a wedge-shaped block *g* being placed above them to keep them at a proper angle. The planks *h* that protect the sides of the heading are termed **lagging**. The flooring *k* serves to exclude the liquid mud, which would otherwise be forced from underneath by the external pressure. The horizontal cross-timber *l*, as well as the longitudinal timbers *m* and *n*, are called **struts**. The floor *o* serves as a footing for the workmen while driving the poling boards.

31. If the material penetrated is wet enough to run, it is necessary to constantly maintain a bulkhead of planks *p*, Fig. 33, called **face boards,** held in place by struts *q*. As the poling boards are driven forwards, the top face board is removed, allowing the released material to flow into the gangway. This forms a cavity in the face of the heading, and immediately another bulkhead is started by placing a face board *r* in advance of the boards at *p*, with a strut *s* to keep it in place. When the heading is advanced half the length of the poling boards, a new set of timbers is put in place, the collar of which takes the strain from the poling boards, which would otherwise be soon broken by the great pressure above them.

As the section is enlarged, other timbers are substituted, until the complete section is excavated. The masonry lining should follow immediately. The less important timbers may be removed as the masonry advances; but the main supports should remain in place; the masonry should be built around them, and they should not be disturbed until the arch is keyed. They can then be removed with safety, and the vacancies in the masonry carefully filled and grouted. All open spaces back of the masonry should be filled with well-rammed concrete.

32. Measuring Excavation.—Various methods are employed for determining the cross-section of the tunnel in order to compute the volume excavated. The best device is the following, illustrated in Fig. 34: A semicircular protractor *a b* with a diameter of from 8 to 10 feet, and made of light pine, is set up at right angles to the center line of the tunnel. The diameter *a b* of the protractor is brought into a horizontal position by means of the spirit level *c*, and placed at any desired height above the floor of the tunnel. It is not necessary that the center *d* should be on the center line of the tunnel, but its exact position with reference to that line should be very accurately determined. A sliding rod *d e*, one end of which is fastened to the center *d* of the protractor, measures the distances to the tunnel walls

on radial lines. The angles that these lines make with the horizontal are read directly from the protractor. The tunnel section and the actual working measurements are then platted on cross-section paper, from which the area of the cross-section can be readily calculated, or it can be measured with a planimeter. Cross-sections are taken at intervals of

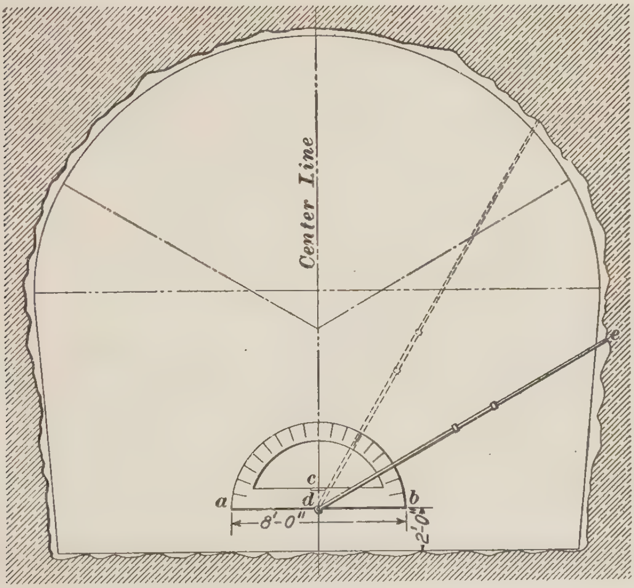

FIG. 34

about 10 feet. As the distance between them is so small, and as, besides, the areas are nearly equal, the volume between any two of them is computed with sufficient accuracy by the method of averaging end areas.

TUNNEL LINING

33. Requirements.—Some kinds of rock are so hard and firm and so unaffected by exposure to the air that they may be safely left in their natural state without any kind of lining. Where lining is necessary, almost the only suitable material is masonry in some form. The character and

thickness of the lining are purely matters of judgment and, unfortunately, of judgment that is little better than guesswork. Although a reasonably close estimate may be made of the capacity of a given form of lining to withstand a given stress, the amount and character of the stresses to which a tunnel lining will be subjected are frequently matters that can at best only be roughly approximated. If a tunnel is merely a subsurface tunnel carrying a definite load on the surface, the problem is comparatively simple; but when, as sometimes occurs, the tunnel passes through strata of soft material, which are themselves in a state of unstable equilibrium, there is a tendency to distortion and crushing by forces that cannot be accurately known or measured. The only method to be followed in such cases is to adopt a lining that is probably strong enough, and to watch the tunnel carefully for any evidence of settlement or failure.

34. Timber and Masonry Lining.—Timber is frequently used as the so-called permanent lining of a tunnel, but it hardly deserves the name of permanent, since it cannot be expected to last indefinitely. If the outside of the tunnel were perfectly dry, as is the case with the framing timbers in the roof of a house, or if, on the contrary, the timbers were constantly wet, as they would be under water, they might be expected to last almost indefinitely; but, in a tunnel, neither of these conditions obtains, the timber being subject to constant alternations of wetness and dryness, which promote rapid decay. A timber lining should therefore be constructed with the idea that it will be renewed or replaced by a masonry lining. Sometimes, the tunnel is excavated with such a width that, even after the timber lining is put in, there is sufficient space to build a masonry lining covering the timber and still have sufficient clearance room for the cars. This, however, is bad practice, especially when the ground is soft and treacherous, for in time the timber will decay utterly and permit the earth above the tunnel to drop down on the masonry lining. The impact of a great mass of soil is likely to develop a high pressure that

may cause serious damage to or even crush the masonry lining. A better way to substitute a masonry for a timber lining is to take out all the timber, even the cordwood that is frequently used to fill up the irregularities caused by the blasting, and to substitute a masonry lining with a backing of concrete to fill up all gaps.

35. Arch Masonry Lining.—When the earth pressure becomes very great, it may be necessary to put in a masonry lining that is arched, not only at the top, but also on the sides and bottom. In Fig. 35 is shown an illustration of the lining of an Austrian tunnel. The side walls were constructed in an arched form of brick, which rests on a line of stone quoins at the lower corners. The dotted lines across the bottom show how, in especially soft ground, even an inverted arch was considered necessary. The timbering is very heavy. Two headings were made, both in the center line, one at the bottom and the other at the top of the final full section. It should be noted that the brickwork is carried behind the arch line proper, so as to entirely fill the space between the arch and the lagging, the timbers being entirely taken out as fast as the brick lining was sufficiently complete to allow the safe removal of the timber.

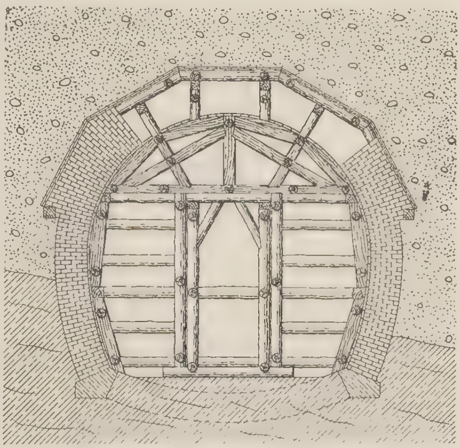

FIG. 35

36. Centers and Molds.—In putting in a masonry lining for a tunnel, forms or patterns are used to show the exact outline of the masonry required. Such patterns vary in number and shape according to the form of lining

and the kind of masonry used. Where the excavation is lined on the bottom, as well as on the sides and roof, the masonry work is divided into three parts; namely, the invert, or floor, masonry, the side-wall masonry, and the roof-arch masonry. A separate pattern is required in constructing each part of the lining; these patterns are called, respectively, *ground molds*, *leading frames*, and *centers*. The shape and construction of each form or pattern will be described.

37. Ground Molds.—A **ground mold** for tunnel masonry consists of a wooden frame or pattern of exactly the form and dimensions of the cross-section of the masonry used for the floor lining. A good form of ground mold is made of plank about 3 inches thick; it is of such shape that its lower edge fits the surface of the excavation, and its upper edge fits the curve of the masonry floor. Such a mold is illustrated in Fig. 36. In order to afford convenience in handling them in a restricted space, ground molds are usually made in two parts, which are joined together in the middle by an iron plate on each side, as shown in the figure. The plates are bolted through from side to side. In using the molds, they are placed at right angles to the axis of the tunnel if the tunnel is straight, and on radial lines if the tunnel is on a curve. They are centered by means of a transit, and leveled until the top of the mold is at exactly the proper elevation. Usually, two molds are employed, at a convenient distance apart, and cords are stretched tightly from one to the other to mark the outline of the masonry. Sometimes, only one mold is employed, in which case the outline of the finished masonry is used instead of another mold.

FIG. 36

38. Leading Frames.—As already stated, the forms or patterns for the side walls are called **leading frames.** They are usually made of plank of suitable thickness; one edge is cut to the curve of the excavation of the side of the tunnel, and the other edge is made to conform to the face of

the side wall. In Fig. 37 is shown a form of leading frame suitable for use in a tunnel whose side walls slope inwards toward the bottom. Leading frames, when in position for use, are commonly set with the lower portion in contact with some fixed part of the ground. The position of the top can be fixed by means of a plumb-line or spirit level, as may be convenient. In Fig. 37 is shown the relative position of a ground mold and a leading frame when set for masonry.

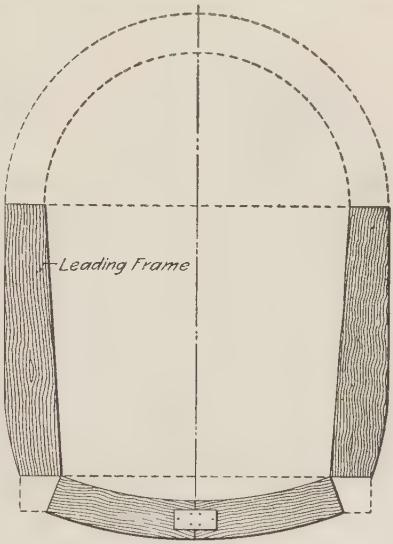

FIG. 37

39. Centers.—The form on which the roof arch masonry of the tunnel is built is called a **center.** Tunnel centers are usually required to support the roof pressure, in addition to the weight of the masonry during the construction of the roof arch, and for this reason they must be made strong. They are generally similar in form to centers used for ordinary arch masonry. In tunnel work, owing to the confined space in which the work must be done, it is customary to omit the center or radial bracing of the arch centers. Where heavy loads have to be supported, the tunnel centers are trussed in such a manner as to leave room for working under the central position. A good form of trussed center is shown in Fig. 38. Where the lining is made of concrete, a special form of center is used, which is mounted on wheels and is moved on a track through the tunnel as the work progresses.

40. Lagging.—The **lagging** consists of narrow strips secured to the outer surface of the arch frames, and spanning the opening between adjacent centers. The outer surface of

the lagging is made to conform exactly to the curve of the roof arch, and forms the mold for the surface of the arch masonry. For stone masonry, the strips of lagging are placed so as to come under the joints between the ring stones, leaving an interval between adjacent strips. For brick and concrete arches, the strips are laid close together. The lagging should be strong enough to hold up the arch masonry, and also to support the short props that keep the poling boards in place after the timber supports have been removed, and until the masonry is completed.

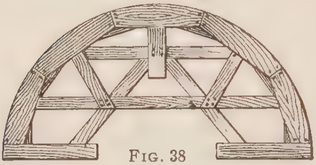

FIG. 38

41. Striking the Centers. The centers are brought to the proper elevation by means of wooden wedges inserted between the bottom of the posts that support the centers and the floor of the tunnel. These wedges are usually made of hard wood, and are about 6 inches wide, 4 inches thick, and 18 inches long. After the masonry is completed, these wedges are withdrawn, allowing the center to drop clear of the masonry; this operation is called **striking the centers.** Usually, the centers are not struck immediately after the masonry is completed, but are allowed to remain a sufficient time to allow the mortar to set hard.

42. Portals.—The name **portal** is applied to the facing that is usually built at each end of a tunnel. Even when the main body of the tunnel is not lined, it is usual to construct a facing of masonry at the end, as the roof of the tunnel is usually rather thin at the ends, and the disintegrating action of the weather on a roof of solid rock might, in time, cause a dangerous breakage. It sometimes happens that the natural surface outside the end of the tunnel is nearly or entirely vertical. When this occurs, the material is usually very solid rock, and it may be safe to leave the portal unlined. If the soil over the mouth of the tunnel is composed entirely or mainly of earth, it is not only necessary to have a lined portal, but it requires unusual care to construct the portal without inducing landslides. Structurally, the portal has

very much the same appearance as the end of a large arched culvert. Above the arch is the usual parapet, from the top of which the earth slopes back, the slope being such as is consistent with stability. The height of the parapet above the arch depends on the depth to which the open-cut approach is made before the tunnel is begun. From each side of the tunnel two wing walls are run out. If the open-cut approach to the tunnel is very long, and is unusually deep at the portal end, the wing walls may run parallel to the line of the road, with a gradually diminishing height. In other cases, side walls are run out from the portal face perpendicular to the line of the road.

The construction of these portals is a serious problem for the engineer. Sometimes it is effected by sinking a shaft at such a distance from the portal that the natural roof of the tunnel is so deep and strong that the tunnel section between the shaft and the portal may be excavated and lined without difficulty. The excavation then proceeds back toward the portal, and by the time the very thin roof immediately over the portal is reached, the lining of the tunnel prevents any serious landslide, and it is comparatively easy to break through. If the portal is constructed by beginning at the outside, there is a danger that the loosening of the earth during the tunnel excavation will cause a landslide that will sweep away the tunnel timbering. This is sometimes prevented by anchoring the timbering immediately back of the portal with a mass of heavy stones, so that, if a landslide occurs, it will not disturb this weighted timbering.

Frequently, the design of tunnel portals is made somewhat elaborate, but this must be considered a matter of architecture rather than of engineering.

DAMS

INTRODUCTION

1. Classification of Dams.—Dams are divided into several classes, according to the material of which they are composed. The principal classes, and those that will be described in this Section, are **masonry dams**, which are built of masonry, either cut stone, rubble, or concrete, or a combination of these materials; and **earth dams**, which are built of earth, with a masonry center wall. Some dams are built of earth alone, but unless the dam is of a very insignificant height, this kind of construction is not to be recommended.

When rock on which the proposed dam can be built is found on the bottom and at the sides of the valley, a masonry dam is always to be preferred. If no rock is found, so that the dam must be built on an earth foundation, an earthen dam, with a masonry center wall, is greatly to be preferred, as being more secure. If, as is frequently the case, rock exists on one side of the valley and not on the other, the dam may be of a composite character; the center wall and earth embankment must be placed on the side where there is no rock foundation.

2. Preparations for Construction.—After a comparison of all the locations examined and studied, both on the ground and on paper, the site of the dam is definitely determined. The next thing is to prepare for construction. For this purpose, the center line of the dam is run out and established by permanent monuments, either of stone, or,

although less advantageously, of strong posts driven into the ground with their heads nearly level with the surface, so that an instrument can be conveniently set over them. Two of such monuments should be set at each end of the center line, perhaps 100 feet apart, making four in all, so that there shall be no danger of losing the line by the destruction or displacement of any two of them. The whole area that is to be in any way worked over in the construction of the dam is then cross-sectioned by dividing it into squares 25 feet or less on a side, and taking the elevations of all the corners, with intermediate points if the topography of the ground requires it. The squares with the elevations of their corners above datum are mapped in duplicate or even triplicate. One copy should be kept in a place of perfect security, as this map constitutes a record of the ground before any work was commenced, and from it all the excavation and embankment—in a word, all the work that changes the face of the ground—are calculated for the partial and final estimates. The importance of taking and securing this record must not be underrated, as it is the only way to know what work has actually been performed, and to settle disputes that may arise regarding such work. This cross-sectioning will require a great number of stakes to be driven. As these stakes are not permanently preserved, they may be of very light material.

At least two permanent bench marks are established in convenient locations, and, by means of these, temporary ones can be set as they are needed for the work.

EARTHEN DAMS WITH MASONRY CENTER WALLS

CENTER WALL

3. Purpose.—In earthen dams, the **center wall** is the first thing to be considered. This wall should be carried down to a water-tight foundation, if possible, and its ends should be deeply embedded in the sides of the valley. Its object is two-fold: first, to afford an impermeable and indestructible cut-off to any water that might otherwise percolate through the bank, either because the bank itself is not impermeable, or because it has been perforated by musk-rats or other burrowing animals; second, to afford a means of making water-tight connections for the culverts or pipes used for conveying water from the reservoir. Without such water-tight connection as the cut-off wall offers, there is always danger that the water may flow outside of these pipes, form a channel in the bank, and finally cause the destruction of the dam. Several instances of failures from this cause are on record.

4. Foundations.—In building the center wall, the first consideration is the foundation. The wall should be carried down until a stratum is encountered that appears to be impermeable. Generally speaking, the finer the material, the better it is as a foundation. Fine gravel or sand is probably the best; but even when these are mixed with clay, they form an excellent material on which to found the center wall. Fine sand with smooth, round grains, mixed with a large percentage of clay, constitutes what is known as **quicksand,** and when found at a certain depth below the surface, so that it cannot escape laterally, forms a good water-tight foundation. Loose, coarse gravel containing

large stones or cobbles is about the worst material that can be encountered. When such material is found, the center wall must be carried down to a much greater depth than would otherwise be necessary. Increased depth of foundation compensates to a considerable extent for poor quality of material.

5. Dimensions.—The center wall should be carried up as high as the level of the highest water in the reservoir. Its thickness at the surface of the ground should be one-quarter of the height, and this thickness may be reduced by a batter of 1 inch to the foot on both sides, or, preferably, by building the wall with vertical sides and stepping in 2 feet (1 foot on each side) every 10 feet, which amounts to about the same thing. As the foundation ascends the sides of the valley, it is stepped up, but care must be taken to keep the bottom of the wall well below the surface of the ground and well embedded in good material. The ends of the wall must also be carried well into the banks forming the sides of the valley.

6. Puddle Walls.—Sometimes, the masonry center wall is replaced by a puddle wall. Puddle is a mixture of clay and gravel or sand, which are well mixed together, carefully moistened, and rammed in place. When well made of selected materials and properly placed, it makes a safe and satisfactory wall, and is somewhat cheaper than masonry. This class of construction is general in England, where the work is thoroughly understood, and carried to a high degree of perfection. It is very much less common in the United States, where the masonry center wall has become typical of American practice. Since the puddle wall requires very careful selection of materials and many minute precautions in their preparation and placing, the work must be performed by skilled labor. For these reasons, a masonry wall is generally to be preferred.

7. Concrete Walls.—Concrete can be advantageously employed for the construction of the center wall. A center wall built of this material is very strong, because of its

monolithic character; and, as the concrete work is done by machinery and unskilled labor, it is cheaper than a masonry wall.

EMBANKMENT

8. Form and Material.—The center wall, as described, forms the **core** about which the **earthen embankment** is formed. This embankment rises to a certain height above the top of the wall, which height depends on circumstances to be considered hereafter. The embankment is flat on the top, and has a gentle slope on each side, the rate of slope depending greatly on the material of which the bank is formed. The best material is a fine gravel or coarse sand, such as is suitable for making mortar. Clay, although perfectly water-tight when confined, is a treacherous material in a bank, because it is so fine that it actually dissolves in the water, and is liable to be washed away in the form of semifluid mud. Its cohesiveness is also a very objectionable feature; for, if a cavity is formed in a clay bank, it remains open indefinitely, whereas the tendency of a bank composed of sand and gravel is to close all such openings. Thus, if a bar is forced into a mass of clay and then withdrawn, it leaves a round hole, the size of the bar; but if the same thing is done in a bank of fine gravel or sand, the space occupied by the bar is immediately filled up by the material, when the bar is withdrawn.

9. Method of Building.—The earthen embankment should be carried up in horizontal layers, and kept constantly moist by sprinkling. In many specifications, it is provided that these layers must be consolidated with a roller. This is of doubtful expediency, for the tendency of rolling is to form separate and distinct layers or "partings" in the bank. If the material is brought to the bank in wagons, and evenly spread by shovels and horse scrapers, being kept constantly moist meanwhile, the travel of the men, horses, wagons, and scrapers is generally all that is needed to secure a good bank.

Before placing the embankment, the natural surface on

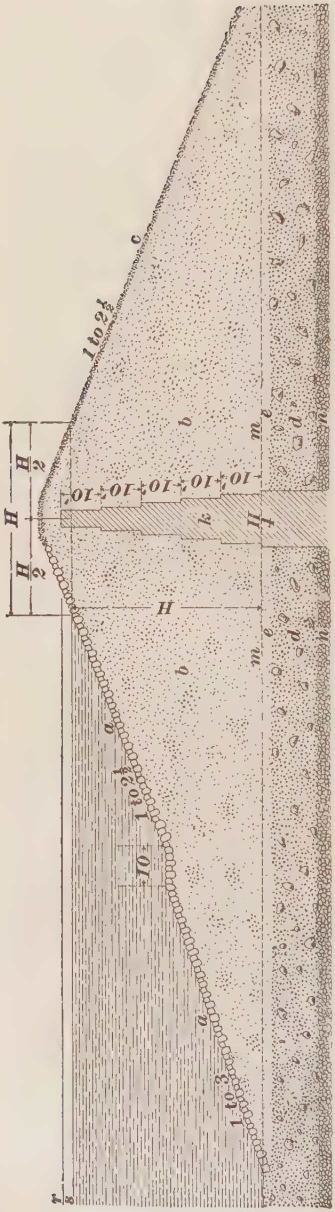

Fig. 1

which it is to stand must be carefully stripped of all sods, roots, and vegetation, so that the first layer of the earthen embankment may rest on and incorporate itself with clean earth. This is particularly necessary under the inner slope.

10. Slope.—A fair average for the outside slope, that is, the slope on the lower or down-stream side *c*, Fig. 1, is 1 vertical to $2\frac{1}{2}$ horizontal. A somewhat flatter slope is advisable on the inside or water side *a* of the bank, say 1 vertical to from $2\frac{1}{2}$ to $3\frac{1}{2}$ horizontal. In a high embankment, say 50 feet or more, it is better to divide the inside slope into two or more steps, as shown in Fig. 1. From the top, a slope of about 1 to $2\frac{1}{2}$ is carried down for 25 or 30 feet, and then a *berm* 8 to 10 feet wide is introduced. From this point, the slope is continued with a somewhat flatter grade, as 1 to 3, for another 25 or 30 feet, when another berm is introduced, and the slope is again continued, either with the same grade as before or, preferably, with a grade somewhat flatter, as 1 to $3\frac{1}{2}$.

The inside slope, next to the water, should be carefully paved, or ripraped with stone for a thickness of from 1 foot to 2 feet, from the bottom to a point well above the high-water line. The greatest thickness should be between the first berm, as above mentioned, and the top of the dam, for that part is most exposed to wave action. The stone composing this paving must be placed and packed by hand, not dumped at random. The outside slope should be sodded, or at least sown with grass and carefully tended until a good sod is formed.

11. Designs of Dams.—Fig. 1 shows all the constructive features in a general way, and represents a good type of earthen dam. In this figure, a is the inner slope of the embankment; b, b, the earthwork embankment; c, the outer slope; and d, d, the natural loose material under the embankment from which the vegetable soil has been stripped, as shown by the lighter shading at e, e. The original surface of the ground is shown by the dotted line m, m; k is the center wall, carried well down into the compact sand and gravel n, n. The line r shows the high-water level, or level of freshet overflow, and s is the low-water level, or level at which overflow begins. It will be perceived that several of the dimensions depend on H, which is the vertical distance from the natural surface of the ground to the level of the overflow, or lip of the spillway. If the dam crosses a deep and wide stream, H must be taken equal to the vertical distance from the bottom of the stream to the lip of the spillway.

The thickness of the earth embankment at the level of the lip is taken equal to H. The earth embankment is evenly divided by the center line of the dam. The center wall is given a thickness at the base equal to $\frac{H}{4}$, and at every 10 feet it is stepped in 1 foot on each side. There is also a small offset, or footing, given to the foundation, which is carried down to a secure formation of fine sand and gravel. The top of the center wall is carried up to the height of high-water level in the reservoir, or to a height equal to $H + D$, D being the depth of notch of the spillway.

The following illustrative example will show how the above principles are applied:

12. Referring to Fig. 1, let $H = 48$ feet, and $D = 2.5$ feet. The center wall commences with a thickness of 12 feet. At a height of 10 feet, it is drawn in by offsets to 10 feet; at 20 feet, to 8 feet; at 30 feet, to 6 feet; and at 40 feet, to 4 feet, which thickness is carried through to the top. The total height of the center wall above the foundation is $48 + 2.5 = 50.5$ feet. The embankment is carried up 8 feet above the level of the spillway; at the level of the spillway, or low-water line, it has a thickness of 48 feet, the top width being 8 feet. At a depth of 27 feet below the spillway, a

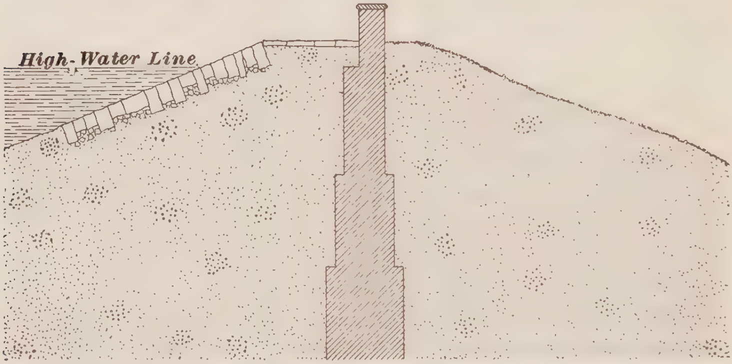

Fig. 2

berm 10 feet wide is introduced, and the slope is continued at the rate of 1 to 3. These dimensions and proportions represent safe minimum values under ordinary conditions; they will be found to furnish a sound basis for the designs to which they apply, subject to modification for local conditions.

13. Another Design.—An excellent, though somewhat more expensive, design for the top of the dam is shown in Fig. 2. In this form of dam, the center wall is extended through the embankment and forms a parapet wall along the top. If the top of the bank is flagged, as shown in the figure, the work is very complete and secure. The top of the bank should slope away from the wall on both sides, to

prevent water from working down between the bank and the wall. The possibility of such an occurrence is perhaps an objectionable feature of this style of construction, which otherwise has much to recommend it.

SPILLWAY, OR OVERFLOW

14. The **spillway** is one of the most important features of a dam. It is the means by which the surplus water, when the reservoir is full, is allowed to run to waste, and want of sufficient discharging capacity in this particular has been probably the most common cause of destruction of earthen dams. If such dams are once overtopped by a flood, especially when not provided with a proper center wall, they are rapidly cut down and destroyed by the water running over them.

Frequently, a natural overflow can be found in some lateral depression of the ground, by which the surplus water can be passed into another valley. When such an overflow occurs, particularly if the depression is in rock, it is generally used, since, besides being conducive to safety, it saves a considerable expense. Sometimes, it is possible to form such an overflow by cutting into some rocky ridge leading either into another valley or into the same valley, lower down, across which the dam is built. In the majority of cases, however, it is found necessary to provide a special piece of masonry construction for a spillway. The spillway is then called a **waste weir.**

15. Dimensions.—The dimensions of a spillway must be proportioned to the amount of water likely to go over in times of freshet. It may be comparatively long and shallow, or short and deep. A convenient length is given by the formula

$$L = 20\sqrt{A}$$

in which L = length, in feet, of lip of spillway;

A = area, in square miles, of watershed above the dam.

Having determined the length, the depth is computed by the weir formulas given in *Hydraulics*. In applying these

formulas, a careful determination must be made of the amount of water to be discharged.

16. Quantity of Water to be Provided For.—If dams are already built in the stream under consideration, or in neighboring streams of nearly the same volume, an examination of their overflows, or spillways, will afford valuable information in designing the new work. If there is a bridge under which all the water of the stream passes, some conception of the amount of flood discharge to be provided for may be obtained by a knowledge of the greatest height to which the water has been observed to rise on the abutments. Otherwise, the discharge, or run-off, can be approximately computed by the Buerkli-Ziegler formula, given in *Sewerage*.

In sections of the country where heavy ice is formed in winter, the depth of the spillway should be sufficient to float the thickest ice that may be formed in the reservoir; otherwise, should a freshet occur when the reservoir is filled with ice, the spillway may be blocked with the broken cakes carried down by the wind and current.

17. Time as a Dimension Factor.—It is clear that the duration of a storm exercises a marked influence on the height to which the water will rise in the reservoir, and consequently on the depth D of the head of water going over the spillway. The time t, in seconds, required for the water to rise to a certain height H_1 over the lip of the spillway, while at the same time discharging over it, may be determined approximately by the following empirical formula, which applies when H_1 is from two-thirds to three-fourths of the total height D to which the given flood may rise:

$$t = \frac{A_1 H_1}{Q_1 - 2 \times L \times H_1 \sqrt{H_1}}$$

in which A_1 = average area of the water surface in the reservoir, in square feet;

L = length of spillway, in feet;

Q_1 = cubic feet of run-off entering the reservoir per second.

Example.—Assume the area of the water surface in a reservoir when full to be 10,000,000 square feet. If the length of the spillway is 155 feet and the discharge is 7,320 cubic feet per second, how long will it take the water to rise to a height of 4 feet?

Solution.—Here, A_1 = 10,000,000, H_1 = 4, Q_1 = 7,320, and L = 155; substituting these values in the formula,

$$t = \frac{10,000,000 \times 4}{7,320 - 2 \times 155 \times 4\sqrt{4}} = 8,264 \text{ sec., or 2 hr. 18 min., nearly. Ans.}$$

EXAMPLES FOR PRACTICE

1. The approximate area of a reservoir is 25,000,000 square feet. If the length of the spillway is 150 feet, and the discharge 7,500 cubic feet per second, how long will it take the water to rise to a height of 2 feet in the spillway? Ans. 2 hr. 2 min.

2. The approximate area of a reservoir is 10,000,000 square feet. If the length of the spillway is 90 feet and the discharge 6,800 cubic feet per second, how long will it take the water to rise to a height of 3 feet in the spillway? Ans. 1 hr. 25 min.

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18. Artificial Spillway.—When no natural overflow, as already mentioned, is available, an artificial one must be built. This will generally be located in the line of the stream across which the dam is erected, although the spillway may sometimes be placed at or near one end of the dam, if the character of the ground and other conditions favor this construction. If rock is not found on which to build the structure, the foundations must be carried well down, perhaps deeper than those of the adjacent center wall.

19. Cross-Section of Spillway.—The form and dimensions of the cross-section of the spillway vary greatly; excellent examples from actual structures show considerable difference in the ideas of their designers. There are no fixed rules governing this point. One of the principal differences that will be noticed in existing works is the form of the face of the spillway over which the water passes. This is sometimes formed in steps, and sometimes in a concave curve. When the latter form is used, the face stones should

be cut, at least roughly, to voussoir shape, and their faces, over which the water passes, dressed smooth. In the best work of this class, the facing is of cut stone throughout. This is, of course, very expensive. The object sought in using steps is to break the force of the falling water, so that it will reach the bottom with very little velocity, either vertical or horizontal. The idea embraced in the curved face is exactly the contrary, the object sought—or at least attained—being the greatest possible horizontal velocity urging the water forwards away from the foot of the spillway when it has reached the bottom. Of the two, the step system seems preferable, as the water appears to reach the bottom with less destructive force when its fall is thus broken. Of course, lower down in the stream, the velocity of the water is the same, whichever system is adopted. For low dams, say up to 15 or 20 feet, the face may be nearly vertical, giving the water a clear fall on the apron at the bottom. For higher dams, say up to 60 or 70 feet, the form shown in Fig. 3 is a very good one.

Let AB represent the height from the bottom of the spillway to the level of high water, or freshet flow, in the reservoir. Take AD equal to nine-tenths of AB; also, take AC equal to AD. Join C and D. The face EF has a batter of 1 in 12. Since F, the intersection of EF and CD, is the point where the steps begin, it is the essential point on a working drawing. As the slope of CD is 1 in 1, KF, the horizontal distance of F from AC, is equal to CK, the vertical distance of F below C. But the face batter of GF is 1 in 12; therefore, F is one-twelfth of CK farther from AC than G is; that is,

$$KF = CG + \frac{Ck}{12}$$

or, since $CK = KF$,

$$KF = CG + \frac{KF}{12}$$

The top width CE is made about one-fifth of AC. The face CE is given a batter of about 1 in 4 or 5, and is made of cut stone. The corner E is rounded off.

Spillway sections for still higher dams constitute special cases and can better be considered after a study of high masonry dams.

EXAMPLE.—Referring to Fig. 3, let AB = 70 feet, and let the vertical height of high water over E, the lip of the spillway, be 3.5 feet. To determine the other dimensions.

SOLUTION.—In Fig. 3, $AD = \frac{9}{10} \times 70 = 63$ ft. $CB = 70 - 63 = 7$ ft. Since E is 3.5 ft. below high-water level, it is $7 - 3.5 = 3.5$ ft. higher than C. Now, CE is to be about $\frac{1}{5} \times 63 = 12.6$ ft. But this dimension admits of a good deal of latitude, and if CG is made 13 ft., CE, though less than 12.6 ft., will be long enough. Hence, CG is taken as 13 ft. Then, $x = CK = KF = \frac{12}{11} \times 13 = 14.2$ ft. This fixes the point F; and from this point, the face EF is drawn with a batter of 1 in 12, up to within 3.5 ft. of high-water level. Ans.

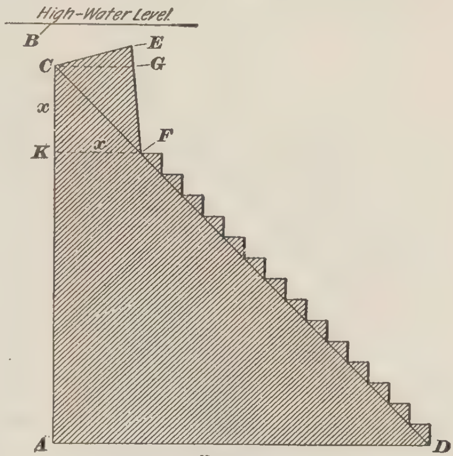

FIG. 3

20. Accessories of Spillway.—The sides of the spillway where it cuts through the embankment must necessarily be protected by wing walls to prevent the earth from falling into it. These wing walls need not have as flat a slope as the exterior embankment, as this would make them unnecessarily large and expensive. They may have a slope of about 1 to $1\frac{1}{2}$, with the bank graded down to them. The top of the wall may have a coping, well doweled into the stones, or be left with the horizontal courses forming steps; the latter form of construction, though not so neat in appearance, is more substantial. When the bed of the stream is not rocky, the foot of the spillway must be protected by an apron composed of very heavy stones, if they can be obtained, laid in cement on a bed of concrete. This apron must be extended well

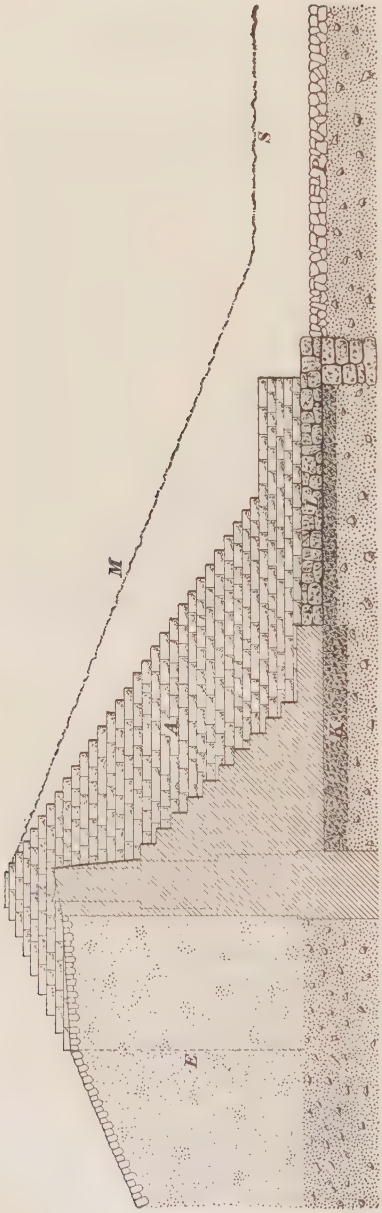

Fig. 4

beyond the foot of the spillway, the distance depending on the height of the dam, the volume of water passing over it, and the nature of the bed of the stream. It should never extend less than a distance equal to the height of the dam. Beyond this, it will be well to protect the bed of the stream between the banks for some distance farther with heavy, dry stone paving. All these features are shown in Fig. 4. No dimensions are given in this sketch, because the conditions of each particular case will greatly modify the details; but the general arrangement and proportions will always resemble those shown. In this sketch, the wing wall of ashlar masonry is shown at *A*; *M* is the exterior slope of the embankment, and *S* the bank of the stream below the dam. The apron *L* is composed of heavy stones, laid in cement mortar on a bed *K* of concrete. There is a cut-off at the end of the apron, which may

advantageously be built of concrete instead of rubble masonry, as shown in the figure. The dry stone paving P, composed of heavy stones, forms a continuation of the apron.

APPLIANCES FOR DRAWING OFF THE WATER

21. Requisites.—There are many devices for the control of the water. Their main requirements are that they be simple and effective and not liable to get out of order. In all cases, a communication must be established between the inside and the outside of the dam, and it is of the utmost importance that no water shall follow along the outside of the appliance—whether tunnel, gallery, or pipe lines—by means of which this communication is established. Herein lies one of the great advantages of a substantial center wall, for it affords the means of making a water-tight connection in the center of the embankment, beyond which no water can trickle. As these features of the dam call for a considerable amount of heavy and expensive masonry, it is also important to design them so that the necessary degree of solidity may be secured with the least possible volume of masonry. This is best accomplished by grouping the different parts together, so that they may be mutually supporting, and so that a portion of one may also form a portion of another.

A system of valves is indispensable for the control of the water. In small reservoirs, these valves may be inserted in the pipe line outside the reservoir and have only an ordinary valve box to indicate their location. Generally, however, they are at the reservoir end of the line, the valve stem thus being below water level, so that a shaft or a tunnel becomes necessary to secure access to the valves.

22. Details of Tunnel Method.—The **tunnel method** is common in European practice, and involves a masonry core wall and a tunnel from the lower face through the embankment, the pipe line being laid in this tunnel. The draw-off pipes are laid through the core wall and are carried down through the tunnel open to view and available for repairs.

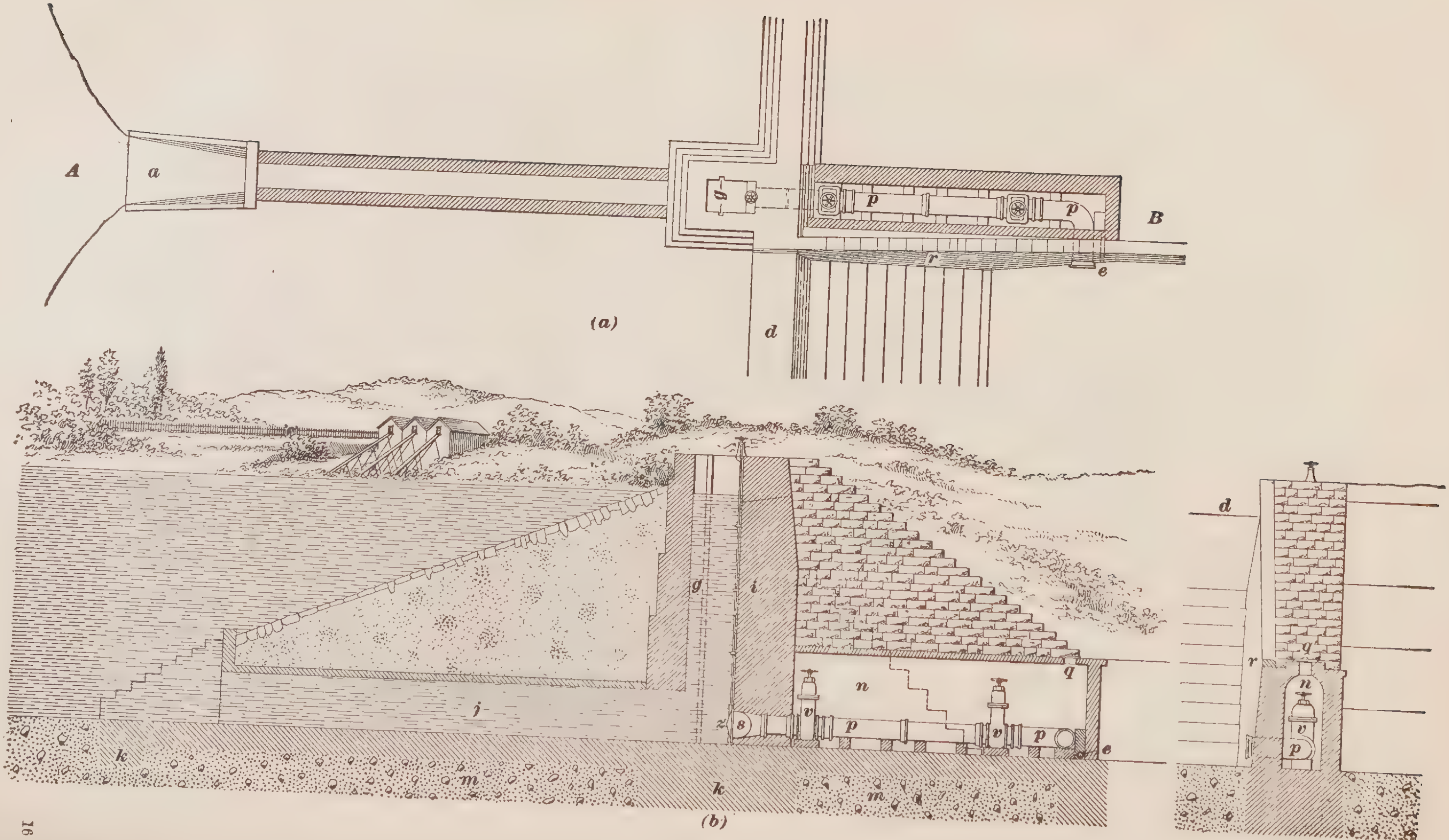

FIG. 5

Fig. 5 (*a*) shows a plan and (*b*) a vertical section on the line *A B* of a general system of design that experience proves to be strong and satisfactory. In the same figure, (*c*) shows a front elevation and section through the gate-house *n*, looking toward the spillway from outside the reservoir, it being assumed that the embankment is removed. On the inside of the dam, adjacent to the spillway *d*, is built a water tower *i*, one side of which is formed by the prolongation of the spillway itself. This tower communicates with the inside of the reservoir by means of an arched gallery or tunnel *j*, which passes under the interior embankment and terminates in an open portal *a* with wing walls. The top of the tower is level with the top of the embankment. The reducer *s* leading to the cast-iron pipe *p* is securely built into the opposite wall of the tower, and the pipe itself is placed in an arched gallery *n* under the exterior embankment; one wall of this gallery forms part of the main wing wall *r* of the spillway. The reducer *s* is a special casting having a rectangular opening at the face, the area of which is equal to or greater than the area of the pipe to which the other end is fitted. The pipe *p* discharges, in some convenient way, into the channel of the stream below the dam. In this way, a clear communication is established between the inside and the outside of the reservoir.

The water passing through this system can be controlled in several ways. The best way is to close the mouth of the reducer on the inside of the tower with a sliding sluice gate *z*, as shown in the figure. Besides this, there should be a stop-cock, or valve, on the pipe inside of the exterior gallery *n*, by which the letting on or shutting off of the water is ordinarily effected. The sluice gate *z* is kept open, and is closed only in case of emergency, such as an accident to the valve. If no such sluice gate is provided, there should be two stop-cocks on the pipe. In the figures, two valves *v*, *v* are shown in addition to the sluice gate, but generally this is not considered necessary.

Whichever of the two systems is employed—and the single-valve and sluice gate is probably the preferable one—there

should always be a set of grooves *g* cut in the masonry of the tower, in which, in an emergency, stop-planks may be placed. **Stop-planks** are heavy timbers of suitable dimensions that are slipped one by one into the grooves; being reeinforced with iron plates, these planks sink readily in the water. The iron plates may be made advantageously in the form of angle irons, so that by means of hooks the plank can be withdrawn. In case it is desired to gain access to the mouth of the pipe without emptying the reservoir, these stop-planks constitute a very valuable means of shutting off the water. Access to the gallery *n* is provided by the manhole *q*, and the gallery is kept free from water by the small drain *e*. The whole structure rests on a hydraulic rubble or concrete foundation *k*, *k*, which must extend down to rock or solid ground *m*, *m*.

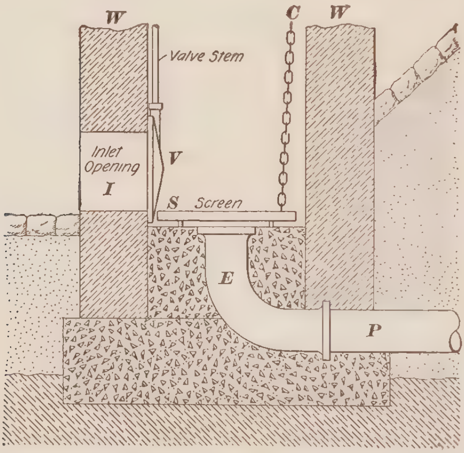

FIG. 6

23. Details of Shaft Method.—In the **shaft method,** the outlet pipe is laid from a point outside of the reservoir on the lower level, through or around the embankment, to a tower or shaft to which the water has direct access. This tower may be either entirely away from the dam, or at the inside toe of the embankment slope; or it may be built shaft-like through the center of the dam, water being brought in by one or more pipes. Fig. 6 shows the simplest construction at the toe. The pipe *P* is brought in under the embankment, and turned up vertically by an elbow *E* ending under a screen *S*, which, to be cleaned, may be lifted by means of the chain *C*. Water enters the tower *W*, *W*, which may be circular or rectangular,

through the inlet opening I, and flows out through E and P. The inlet I is provided with a valve V operated from the floor above by a long valve stem.

Fig. 7 shows a more elaborate design, on the same general plan as that just described. The tower is built, as before, at the inside toe of the embankment, and is connected at the top by a light footbridge; P_1 and P_2 are the outlet pipes, and g, g_1, g_2 are three inlets through the masonry; S, S are a series of rod screens through which

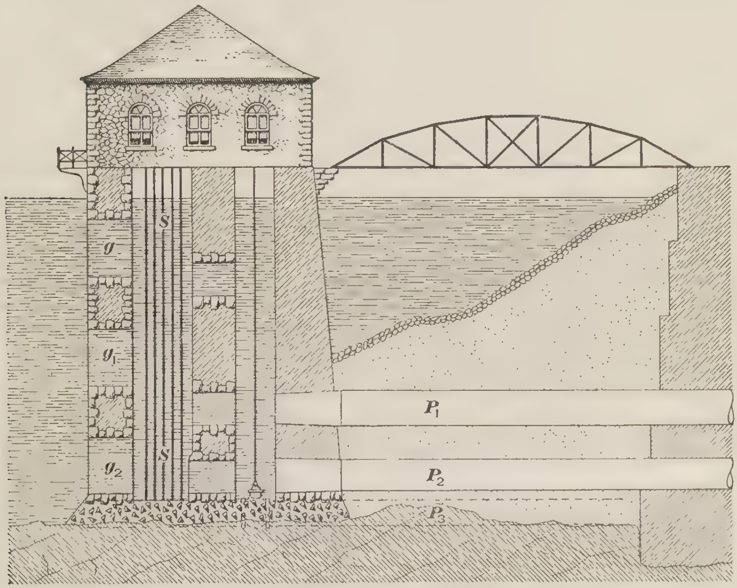

FIG. 7

the water has to pass; P_3 is the drain pipe to be used occasionally, when all the water is to be drawn off. Valves, not shown, are provided for the pipes P_1 and P_2 and at the inlet openings g, g_1, g_2.

Fig. 8 shows the simplest type of the method in which the tower is placed in or near the center of the embankment: W, W are the walls of the tower; g, g_1, the inlet openings at different levels; P, the outlet pipes; and the triangle C, two concrete or masonry walls, between which the water has

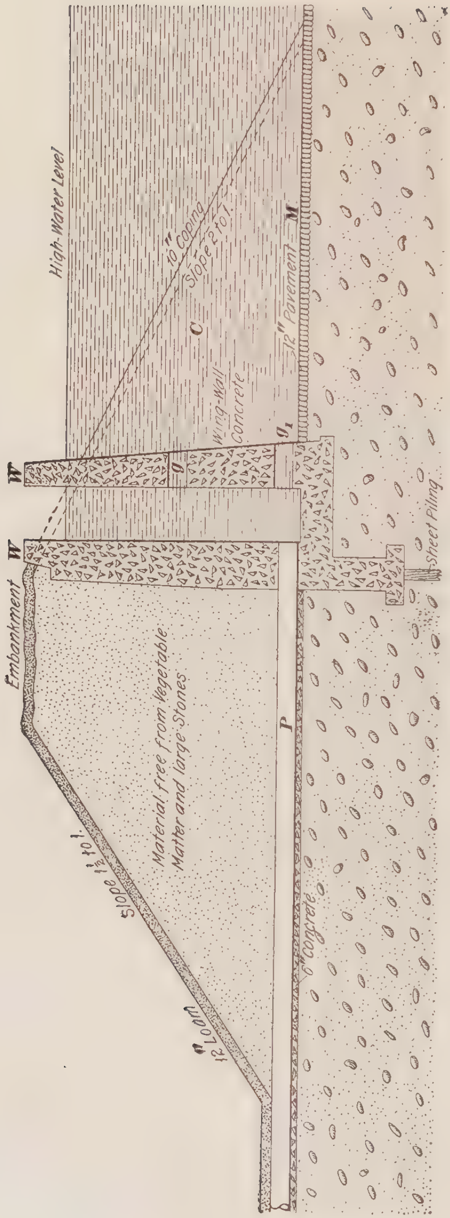
High-Water Level
10" Coping
Slope 2 to 1
C
Wing-Wall Concrete
12" Pavement
M
g_1
g
W
W
Embankment
Material free from Vegetable Matter and large Stones
Slope 1½ to 1
12" Loam
P
6" Concrete
Sheet Piling

FIG. 8

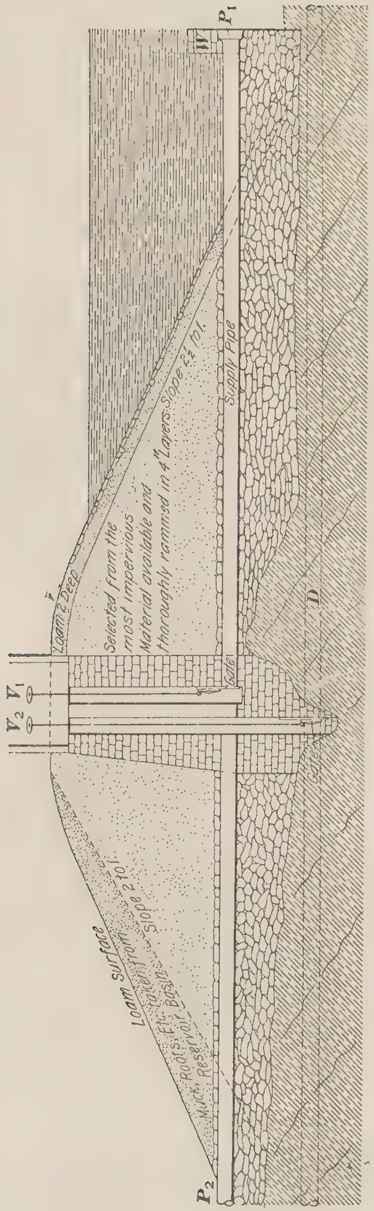

FIG. 9

access to the tower. A pavement M is also provided to prevent washing.

Fig. 9 modifies this last design by substituting for the entrance walls a single entrance pipe, the exit pipe P_2 remaining as before. The figure shows the valve V_1 on the main pipe and the valve V_2 on the drain pipe D. The pipe P_1 has its end protected by a small wall W.

24. Modifications for Small Reservoirs. For small reservoirs, the appliances and arrangements described above are frequently simplified. For instance, the tower, the object of which is to afford means of applying the sluice gates and stop-plank, may be dispensed with, and the pipe run through the center wall to the foot of the interior slope; or, the arched gallery may be continued to the center wall and then connected with the pipe. In these cases, the only control of the water will be by means of the stop-cocks in the exterior gate house.

MASONRY DAMS

LOW MASONRY DAMS

25. General Considerations.—In the structures hitherto treated, there has been no attempt made to determine their dimensions by calculation, as they do not lend themselves to exact mathematical treatment. In earthen dams, all dimensions are fixed by empirical rules; that is, experience has taught that a certain thickness of bank and certain ratios of slope lead to safe results. In masonry dams, the case is different. Given a wall sustaining a certain head of water, it is easy to calculate almost exactly the character, intensity, point of application, and direction of action of the destructive force or forces bearing on it. The resistance of the wall can also be computed with a great degree of approximation. The task of designing such structures is, therefore, more satisfactory than in the case of earthen embankments, and admits of a more scientific course of procedure.

26. Thrust Against Dam.—Expressed in pounds, the horizontal component T of the thrust against any surface $A\,B$, Fig. 10—whether vertical as at (a), inclined as at (b), or curved as at (c)—is the same, and is equal to half the square of the height H or head of water, in feet, pressing against $A\,B$, multiplied by 62.5, which is the weight, in pounds, of a cubic foot of water, and by the length of the surface. The horizontal thrust T_1 per foot of length of the dam is, therefore, given by the formula

$$T_1 = \frac{62.5\,H^2 \times 1}{2} = 31.25\,H^2 \qquad (1)$$

Also, the point of application of this thrust is the same for (a), (b), and (c); namely, at one-third of the height H

from the bottom of the wall. Hence, for the overturning moment M_1 of the horizontal thrust about the point C, we have

$$M_1 = 31.25\,H^2 \times \frac{H}{3} = 10.42\,H^3 \qquad (2)$$

In this formula, M_1, expressed in foot-pounds, is the moment about C of the thrust acting on 1 foot of length of the dam. Although the real resulting pressure acting on the dam is normal to the surface of the latter, and usually

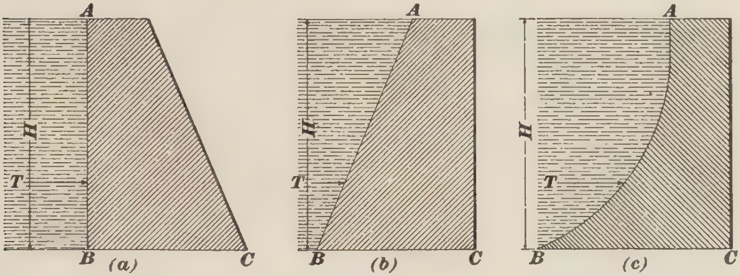

FIG. 10

has a vertical component, this component is generally small, and may be neglected. The error arising from neglecting this component is on the side of safety.

EXAMPLE.—The depth H of water pressing against the curved surface $A\,B$, Fig. 10 (c), is 23 feet 7 inches. (a) What is the intensity, in pounds, of the horizontal thrust per foot of length of wall? (b) What is the overturning moment, in foot-pounds, about the point C, per foot of length of wall?

SOLUTION.— (a) $T_1 = 31.25(\frac{283}{12})^2 = 17{,}380$ lb. Ans.
(b) $M_1 = 10.42(\frac{283}{12})^3 = 136{,}680$ ft.-lb. Ans.

27. Tendency of the Thrust.—The thrust T tends to move the wall in two ways; namely, it tends to push the wall bodily and make it slide horizontally along its base; and it tends to upset the wall by making it turn about the point C, Fig. 10. The first of these tendencies must be resisted by the friction between the dam and its foundation; the second, by the weight of the dam. The action of these resistances will now be explained.

28. Resistance to Sliding.—The frictional resistance that the dam opposes to the tendency of the thrust of the

water to move it forwards on its base is equal to the weight of the dam multiplied by the coefficient of friction between the dam and the material on which the dam rests. Much uncertainty attends the determination of the coefficient of friction; an ordinary estimate places it at about .75. It must be noted that friction only is now considered, as if the dam were merely standing on a level base with no mortar joint intervening. The adherence of the mortar and the bond of the work are both neglected. The resistance thus estimated is, therefore, much below the real resistance. It is at least safe, however, and will here be adopted. As will be presently shown, the tendency of the wall or dam to slide forwards is not, under ordinary circumstances, the destructive force most to be feared.

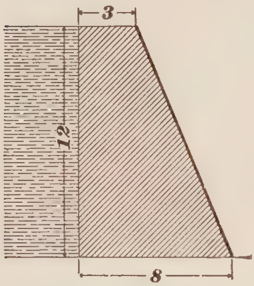

FIG. 11

In all calculations relating to the resistance of such walls as are now being considered, a length of 1 foot, or a slice of the wall 1 foot thick, is always taken, because, since the whole of the wall, if of uniform height, is made up of a succession of such slices, what is true of one will be true of all. The convenience of this consists in the fact that the area of a vertical cross-section of the wall is then numerically equal to the volume of the slice in cubic feet. Throughout all the following calculations, the units of length and weight are, respectively, the foot and the pound avoirdupois.

If R is the resistance of the wall, per foot of length, to sliding, and W is the weight of the wall per foot of length, then

$$R = .75\,W \qquad (1)$$

If the vertical cross-section of the wall is denoted by A, and the weight of the material per cubic foot is denoted by w, formula **1** may be written

$$R = .75\,A\,w \qquad (2)$$

EXAMPLE 1.—A trapezoidal wall, Fig. 11, 12 feet high, 3 feet wide at the top and 8 feet at the bottom, weighs 115 pounds per cubic foot. To determine: (*a*) its resistance to sliding; (*b*) its factor of safety.

SOLUTION.—(*a*) Substituting in formula **2**,

$$R = .75 \times \frac{8+3}{2} \times 12 \times 115 = 5{,}692.5 \text{ lb.} \quad \text{Ans.}$$

(*b*) In order to determine the factor of safety, it is necessary to find the amount of thrust. From formula **1** of Art. **26**,

$$T_1 = 31.25 \times 12^2 = 4{,}500 \text{ lb.}$$

Then, the factor of safety is equal to the resistance divided by the thrust, or 5,692.5 ÷ 4,500 = 1.27. Ans.

EXAMPLE 2.—Let the wall in example 1 be built of granite weighing 170 pounds per cubic foot. Required: (*a*) its resistance to sliding; (*b*) the factor of safety.

SOLUTION.—(*a*) Substituting in formula **2**,

$$R = .75 \times \frac{8+3}{2} \times 12 \times 170 = 8{,}415 \text{ lb.} \quad \text{Ans.}$$

(*b*) The factor of safety is equal to the resistance divided by the thrust, which was determined above. Then, the factor of safety is equal to 8.415 ÷ 4,500 = 1.87. Ans.

29. Resistance to Overturning.—The resistance to overturning, called the **moment of stability** of the wall, is the static moment of the wall with reference to the point about which rotation tends to take place, that is, about the toe *C*, Fig. 12, of the dam; it is equal to the weight of the wall multiplied by the horizontal distance between the toe and a vertical line through the center of gravity of the wall. In order that there may be equilibrium, the moment of stability of the dam per foot of length must be equal to the moment M_1 of the horizontal thrust, as found in Art. **26**.

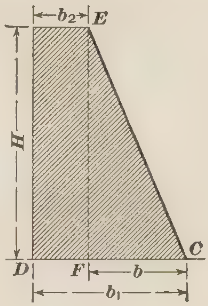

FIG. 12

The almost invariable section or profile of a masonry dam, except for very high dams, is a trapezoid, as shown in Fig. 12. The section may be divided into a rectangle *DE*, whose area is $b_2 H$, and a triangle *EFC*, whose area is $\frac{1}{2}(b_1 - b_2)H$.

The static moment of the rectangle about the toe *C* is

$$b_2 H\left(b + \frac{FD}{2}\right) = b_2 H\left(b_1 - b_2 + \frac{b_2}{2}\right) = b_2 H\left(b_1 - \frac{b_2}{2}\right)$$

The static moment of the triangle about the toe C is, since the center of gravity is horizontally distant $\frac{2b}{3}$ from C,

$$\tfrac{1}{2}(b_1 - b_2)H \times \tfrac{2}{3}(b_1 - b_2) = \tfrac{1}{3}H(b_1 - b_2)^2$$

For the static moment M_a of the whole area, we have, then,

$$\begin{aligned} M_a &= b_2 H\left(b_1 - \frac{b_2}{2}\right) + \tfrac{1}{3}H(b_1 - b_2)^2 \\ &= \frac{H}{3}\left(3\,b_1 b_2 - \frac{3\,b_2^2}{2} + b_1^2 - 2\,b_1 b_2 + b_2^2\right) \\ &= \frac{H}{3}\left(b_1^2 + b_1 b_2 - \frac{b_2^2}{2}\right) \end{aligned}$$

The moment of stability M_s of the dam, per foot of length, is $w\,M_a$, denoting by w the weight of the material per cubic foot. Replacing M_a by its value, we have,

$$M_s = \frac{w\,H}{3}\left(b_1^2 + b_1 b_2 - \frac{b_2^2}{2}\right) \qquad (1)$$

In order that the dam may be secure, M_s must be at least equal to M_1; hence, the least moment of stability that is consistent with equilibrium is given by the formula (see Art. 26) $\frac{w\,H}{3}\left(b_1^2 + b_1 b_2 - \frac{b_2^2}{2}\right) = \frac{31.25\,H^3}{3}$;

whence,

$$H = \sqrt{\frac{\left(b_1^2 + b_1 b_2 - \frac{b_2^2}{2}\right)w}{31.25}} \qquad (2)$$

Example.—What is the moment of stability of the trapezoidal wall of example 1, Art. 28?

Solution.—Here $b_1 = 8$, $b_2 = 3$, $w = 115$, and $H = 12$. Substituting in formula 1,

$$M_s = \frac{115 \times 12}{3}\left(8^2 + 8 \times 3 - \frac{3^2}{2}\right) = 38{,}410 \text{ ft.-lb.} \quad \text{Ans.}$$

30. Design of the Profile.—In designing masonry dams, the height H is determined by the amount of water to be stored, taken in connection with the nature of the place in which the reservoir is built; the weight w is given by the material of which the dam is built; the top width b_2 and the factor of safety are assumed. From these data, the width of base b_1 can be calculated.

Consider first the resistance to sliding. If a factor of safety j is used, formula **1** of Art. **26**, for the horizontal thrust, becomes $T_1 = 31.25\, j H^2$.

The resistance to sliding, from formula **2** of Art. **28**, is, writing $H\left(\frac{b_1 + b_2}{2}\right)$ for the area,

$$R = .75\, H\left(\frac{b_1 + b_2}{2}\right) w.$$

Since R must equal T_1, we must have

$$31.25\, j H^2 = .75\, H\left(\frac{b_1 + b_2}{2}\right) w;$$

whence, $$b_1 = \frac{83.33\, j H}{w} - b_2.$$

EXAMPLE 1.—The height of a trapezoidal wall is 30 feet; width at top, 6 feet; weight per cubic foot, 140 pounds; and factor of safety, 2.5 To determine the required bottom width of the wall to resist sliding.

SOLUTION.—Here, $j = 2.5$, $H = 30$, $w = 140$, and $b_2 = 6$; substituting these values in the last equation,

$$b_1 = \frac{83.33 \times 2.5 \times 30}{140} - 6 = 38.64 \text{ ft.} \quad \text{Ans.}$$

31. To determine the breadth of base of a trapezoidal wall to resist overturning, the moment of stability given by formula **1** of Art. **29** must be equal to the moment of thrust given by formula **2** of Art. **26**. Introducing the factor of safety j, formula **2** of Art. **26** becomes

$$M_1 = \frac{31.25}{3} j H^3$$

Making the moment of thrust equal to the moment of stability,

$$\frac{31.25}{3} j H^3 = \frac{w H}{3}\left(b_1^2 + b_1 b_2 - \frac{b_2^2}{2}\right)$$

Solving for b_1,

$$b_1 = \frac{1}{2}\sqrt{\frac{125\, j H^2}{w} + 3\, b_2^2} - \frac{b_2}{2}$$

EXAMPLE.—The dimensions of a trapezoidal wall and the factor of safety being the same as those given in the example of the preceding article, determine the width of base to resist overturning.

SOLUTION.—Substituting the given values in the formula,

$$b_1 = \frac{1}{2}\sqrt{\frac{125 \times 2.5 \times 30^2}{140} + 3 \times 6^2} - \frac{6}{2} = 20 \text{ ft.} \quad \text{Ans.}$$

EXAMPLES FOR PRACTICE

1. The depth of water pressing against a dam is 75 feet. (*a*) What is the horizontal thrust per foot of length of dam? (*b*) What is the overturning moment, in foot-pounds, about the outer toe of the dam, per foot of length of dam?
Ans. $\begin{cases} (a) & 175,780 \text{ lb.} \\ (b) & 4,395,900 \text{ ft.-lb.} \end{cases}$

2. A trapezoidal wall 25 feet high is 4 feet wide on top, 12 feet at bottom, and weighs 130 pounds per cubic foot. What is its resistance to sliding?
Ans. 19,500 lb.

3. What is the moment of stability of the wall in example 2?
Ans. 199,330 ft.-lb.

4. The height of a trapezoidal wall is 50 feet; width at top, 8 feet; weight per cubic foot, 170 pounds; and factor of safety, 2.5. What is the required bottom width to resist: (*a*) sliding? (*b*) overturning?
Ans. $\begin{cases} (a) & 53.3 \text{ ft.} \\ (b) & 30.6 \text{ ft.} \end{cases}$

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32. Remarks on Stability.—In examining the results given by the formulas of Arts. **30** and **31**, it is evident that a much greater width of base is required to insure security against sliding than is required to guard against overturning. It must be kept in mind, however, that, as already stated, only the mere friction of stone on stone is taken into account when calculating the resistance to sliding; that is, only such friction as might occur if two level surfaces of stone were brought in contact. When it is remembered that a well-bound piece of masonry is by no means in this condition, but is knit together in a more or less homogeneous mass, it will be seen that the tendency to move forwards is counteracted, not by mere friction alone, but also by the resistance to shearing of the stonework. This is a very strong combination, and makes the total resistance so great that experience proves that, when a dam is safe against overturning, it is safe against being moved forwards bodily on its base. If, however, the whole dam were placed on a smooth surface, such as a timber grillage, particularly if the planks were laid in the same direction as the pressure, or if it rested on yielding clay, with only a small depth of foundation, very serious doubts might exist as to whether

it would remain immovable, and careful examinations and calculations would be necessary. In such cases, the coefficient of friction may fall considerably below .75. As, however, all masonry dams should stand on a rock foundation, into which the footing course is well embedded, no danger of their moving bodily forwards need be apprehended if the stability is satisfactory as regards overturning.

33. Average Dimensions.—Calculations made with various practical values for w and b_2 show that a bottom width equal to from $\frac{2}{3} H$ to $\frac{3}{4} H$ will always give a satisfactory factor of safety, and in nearly all cases the smaller of these two values, that is, $b_1 = \frac{2H}{3}$, will give a perfectly secure profile.

HIGH MASONRY DAMS

34. General Considerations.—The trapezoidal profile hitherto considered is the one almost universally adopted for masonry dams up to 50 or 60 feet in height. Beyond this limit, it would no longer be economical nor, in very high dams, practicable. So far, only resistance to sliding and

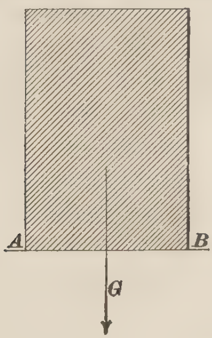


FIG. 13

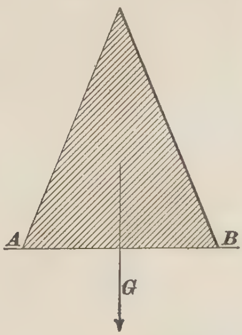


FIG. 14

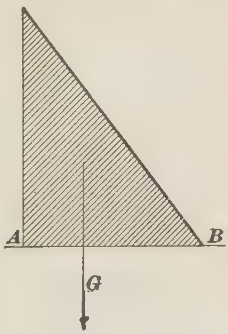


FIG. 15

overturning has been considered, but in very high dams another element of destruction must be taken into account; namely, the crushing of the material under its own weight. In the case of symmetrical figures, the amount of pressure per square unit of base is obtained by dividing the whole

weight resting on the base by the number of square units in the base. Thus, in Figs. 13 and 14, if W represents the total weight of the mass above the base AB, the uniform weight borne per square unit of the base is $\frac{W}{AB}$. If, however, the profile is that shown in Fig. 15, since the figure is not symmetrical about the line G passing through its center of gravity, the weight is not uniformly distributed over each square unit of base, and it is impossible, merely at sight, to say what the maximum intensity of stress may be. All that is clearly evident is that the intensity of stress must be greater on the shorter of the two segments into which the line of action G of the weight cuts the base.

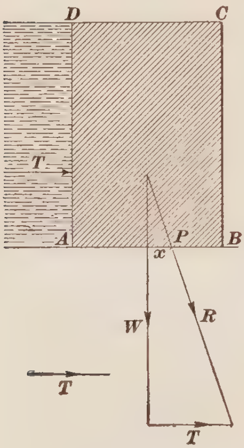


Fig. 16

Even when the profile is symmetrical, the action of an external force may cause the resultant pressure on the base to move from its central position toward one extremity of the base. Thus, in Fig. 16, $ABCD$ represents the profile of a rectangular wall sustaining water pressure; the total pressure R on the base is the resultant of the weight W of the structure and the thrust T of the water. The line of action of R does not intersect the base at the center, but at a point P whose distance x from the center can be readily determined by the principles of statics.

35. Unequally Distributed Load.—Suppose AB,

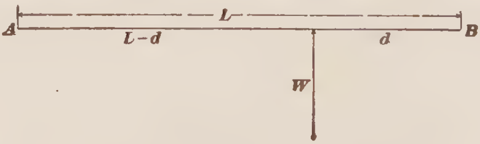


Fig. 17

Fig. 17, to be the base, or any given horizontal course, of a mass of homogeneous masonry. Let W be the vertical

component of the resultant of its weight and any exterior force acting on it; let the line of action of this component divide the length L into two segments d and $L - d$, as shown. The maximum intensity of stress is in the shorter segment d, and its amount may be determined by the following formulas, which are derived by the use of advanced mathematics:

$$P = \frac{4\,W}{L^2}(L - 1.5\,d) \qquad (1)$$

$$P = \frac{2\,W}{3\,d} \qquad (2)$$

In these formulas, P is the maximum intensity of stress, or maximum pressure per unit of area, on the surface $A\,B$. Formula **1** is to be used when d is equal to or greater than $\frac{L}{3}$, and formula **2** when d is equal to or less than $\frac{L}{3}$. When $d = \frac{L}{3}$, the two formulas give the same results. Formula **1** is, with a slight change of notation, given in *Foundations*, Part 1.

Good authorities advise that formula **2** be used with great judgment, and only when d is but little less than $\frac{L}{3}$. Others consider the results obtained by formula **2** to be too small when the obliquity of the resultant of the weight of the mass and the thrust of the water exceeds a certain amount, the error increasing with the obliquity. A preferable formula, though one that has not yet been generally accepted, perhaps because it is not well known, is the following:

$$P = \frac{W(L - d)}{L\,d} \qquad (3)$$

This formula applies to all values of d, and will be used throughout this Section. It gives very nearly the same results as formula **1** for cases in which the latter applies. The three formulas agree for $d = \frac{L}{3}$. For values of d less than $\frac{L}{3}$, formula **3** gives pressures increasingly greater than formula **2**, which, to conform to the ideas of some good

authorities, it should do. It will be explained later, however, that in designing the profile of a masonry dam, whether high or low, the point where the resultant pressure cuts the base should always be kept within the "middle third" of the base, and as near the center as possible. That is, d should always lie between $\frac{L}{3}$ and $\frac{L}{2}$, and the nearer the latter, the better.

36. Conditions of Stress.—In the study of the stresses sustained by a high masonry dam, two conditions are to be considered: the stress when the dam is supporting only its own weight, and the stress when the dam is under the action of the water pressure, in addition to its own weight. These two conditions correspond, respectively, to an empty and a full reservoir. In the former case, the resultant pressure is between the center of the base and the inside toe of the dam; in the latter case, the resultant pressure is between the center and the outside toe.

In designing the proper profile, therefore, it is necessary to give it such a form that, if a line is drawn through it at any point parallel to the base and the resultant of the forces acting on the mass above such line is determined both with and without water pressure, the maximum intensity of stress in the shorter of the two segments into which the resultant divides the line will not exceed the working stress of the material.

It is possible to design a profile that will not only fulfil the above requirement, but will also be exactly, or very nearly, a profile of "equal resistance"; that is to say, one in which the maximum stresses will be equal at all elevations. The result, however, would be a profile that could not be adopted for an actual structure, because it would conflict with practical constructive features that are of still greater importance than a profile of equal resistance.

Several formulas have been devised for determining a proper practical profile, but such formulas, even when simplified by many preliminary assumptions, are still very

complicated and of tedious application. Moreover, as the outcome of all the study that has been given to the subject, a certain type of profile has been evolved to which all designs must very nearly conform; so that at the present day it is needless to go through a series of elaborate calculations, the only result of which must be to reproduce the general type already established. It suffices, therefore, in every particular case, to lay down this general outline and test it at certain elevations, as explained in the following general example.

37. General Illustrative Example.—Let it be required to design the profile of a masonry dam 250 feet high above the surface of the ground, the foundation extending to rock lying 100 feet below the surface. The dam will be built of hydraulic masonry, the average weight of which is 140 pounds per cubic foot. The top width of the dam will be assumed to be 20 feet.

The conditions of the design are that the crushing stress at the base at the level of the ground surface shall not exceed 20,000 pounds per square foot, nor 30,000 pounds 100 feet below, at the bottom of the foundation. It is required, moreover, that at a distance of 100 feet from the top the intensity of stress at the back of the dam, when the reservoir is empty, will not materially exceed 16,000 pounds per square foot, and will increase down to the base, where it may reach 20,000 pounds, as already stated. The pressure at the face, when the reservoir is full, shall be less than the pressure at the back when the reservoir is empty. The batter of either face shall not at any point form an angle of less than 45° with the horizontal. This condition is to prevent the weak edge that would result from a flatter slope. In calculating the water pressure, the surface of the water shall be considered as level with the top of the dam.

This example will be solved by commencing at the top and working downwards, considering first a height of 100 feet, and adding 50 feet successively until the total of 250 feet shall have been reached. As already stated, a

length of dam of 1 foot is taken, so that areas in square feet represent volumes in cubic feet.

The first step will be to lay down the right triangle ABC, Fig. 18, whose altitude is 100 feet and base $\frac{2}{3} \times 100 = 66.67$ feet. This is surmounted by the small triangle CDE, of which the base CD is 20 feet, as required. In the actual dam, the face DE would have a slight batter, about 1 inch to the foot, but for convenience of calculation it is here drawn vertical.

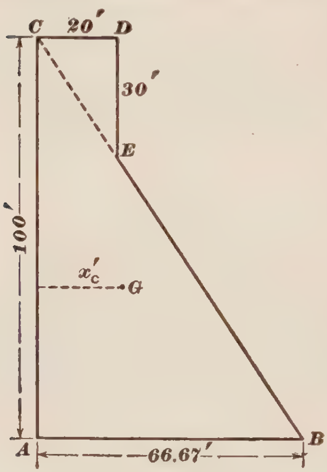


FIG. 18

To find the center of gravity of the figure $ABEDC$, moments will be taken about CA. The distance of the center of gravity of the triangle CDE from AC is one-third the sum of the distances of D and E, which is $\frac{1}{3}(20 + 20) = \frac{40}{3}$, since these distances are each equal to 20. The area of the triangle CDE is $\frac{1}{2} \times 20 \times 30 = 300$ square feet, and, therefore, the moment of this triangle about CA is $\frac{40}{3} \times 300 = 4{,}000$.

Likewise, the distance of the center of gravity of ABC from CA is $\frac{66.67}{3}$, and, as the triangle has an area equal to $\frac{1}{2} \times 66.67 \times 100 = 3{,}333$ square feet, its moment about CA is $\frac{66.67}{3} \times 3{,}333 = 74{,}070$. Therefore, the moment of the total area $ABEDC$ about CA is $4{,}000 + 74{,}070 = 78{,}070$; and, consequently, if the distance of the center of gravity G from CA is denoted by x_c', we must have

$$x_c' = \frac{78{,}070}{\text{area}} = \frac{78{,}070}{300 + 3{,}333} = 21.5 \text{ feet}$$

This distance gives the shorter segment of the base on which the maximum intensity of stress comes when the reservoir is empty. To obtain the intensity of the stress, the number of square feet in the area, 3,633, must be multiplied by

140 pounds, which gives 508,620 pounds for the weight of the mass above the line AB.

The triangle of forces, Fig. 19, is now constructed; it is composed of the vertical line representing the weight of the mass and the horizontal line representing the thrust of the water, which by formula **1** of Art. **26,** is $31.25 \times 100^2 = 312{,}500$ pounds. This thrust intersects the vertical at a distance of $\frac{100}{3}$ feet from the base AB, and thus determines the apex of the triangle of forces from which the weight is laid off to scale, if the graphic method is used. The hypotenuse of the triangle of forces determines by its intersection with the base the shorter segment of 24.67 feet on which the maximum intensity of stress comes when the reservoir is full. This intersection can be determined either graphically or by similar triangles; thus,

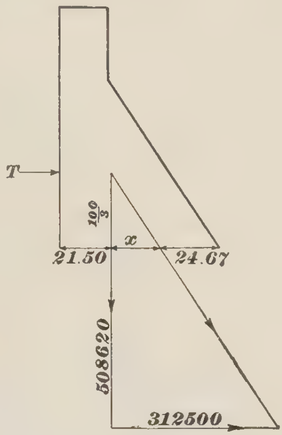


Fig. 19

$$\frac{x}{\frac{100}{3}} = \frac{312{,}500}{508{,}620};$$

whence, $x = 20.5$. Then, the shorter segment is

$$66.67 - (21.5 + 20.5) = 24.67$$

The conformity of this part of the profile with the imposed conditions can now be tested. First, the stress on the segment adjacent to A, Fig. 18, when the reservoir is empty, will be considered. In this case, $W = 508{,}620$, $L = 66.67$, $d = 21.50$, and $L - d = 45.17$. Then, by formula **3** of Art. **35,**

$$P = \frac{508{,}620 \times 45.17}{66.67 \times 21.5} = 16{,}028 \text{ pounds}$$

The maximum intensity of stress on the segment adjacent to B when the reservoir is full will now be found. Here, $d = 24.67$ and $L - d = 42$, and the remaining quantities W and L are the same as before; therefore,

$$P = \frac{508{,}620 \times 42}{66.67 \times 24.67} = 12{,}988 \text{ pounds}$$

It is to be noted that it was not necessary to calculate this last stress, because, since the shorter segment was longer than that corresponding to an empty reservoir, it was certain that the stress would be less, which was all that the conditions of the problem required. It is more satisfactory, however, to work out both stresses fully.

38. In discussing the profile so far, two facts become evident: As regards the limiting stress, when the reservoir is empty, the limit has been practically reached; besides, the line of action of the weight, when the reservoir is empty, passes just outside of the middle third of the base *A B*. A study of Fig. 18 will show that this last fact is due to the influence of the small upper triangle. Had the problem been to construct a profile for a dam 100 feet high only, the base *A B* would have been widened a little to the left of *A*, by giving the back of the dam a slight batter of about 1 horizontal to 4 vertical, commencing at a point about 30 feet above *A*. This would give an increased width to the left of *A* of about 7.5 feet, making the total width of base between 74 and 75 feet, instead of 66.67 feet. This would reduce the intensity of stress on the water side considerably, and on the lower side somewhat. Or, if it were important, owing to the great depth of the foundation, or to some other cause, to keep the base as narrow as possible, and, therefore, to keep the back of the dam vertical, an embankment with a berm followed by a flat slope well ripraped would be placed against the back of the dam, rising to a height of about 30 feet, so as to maintain a constant counter-pressure against the back, and thus reduce the stress.

The reason why no change is made when the profile is the upper part of a much higher dam is that, as will be presently seen, the intensities of stress rapidly increase with the height, necessitating a corresponding widening of the base. It is important, therefore, not to begin widening any sooner than is absolutely necessary.

39. A section of 50 feet is now added to the profile. As already mentioned, it is evident that the stresses will increase

in a more rapid proportion as sections are added to the height; hence, a batter of 20 per cent. will be given to the back and one of 80 per cent. to the face; that is, a triangle 50 feet vertical and 10 feet base is added to the back, and one 50 feet vertical and 40 feet base to the face, as well as the included rectangle, 50×66.67 (see Fig. 20). Throughout this example, $A'C$ will be taken as the axis of moments, distances measured to the right being considered positive, and those measured to the left, negative. The area of the triangle $A A_1 A'$ is $\frac{10 \times 50}{2} = 250$ square feet, and the center of gravity is $\frac{10}{3}$, or 3.33, feet to the left of the axis of moments. The area of the rectangle $A B B' A'$ is $50 \times 66.67 = 3{,}333$ square feet, and the center of gravity is $66.67 \div 2 = 33.33$ feet to the right of the axis of moments. The area of the triangle $B B' B_1$ is $\frac{50 \times 40}{2} = 1{,}000$ square feet, and the center of gravity is $\frac{1}{3}(66.67 + 66.67 + 106.67) = 80$ feet to the right of the axis of moments. The static moment of the part $A B B_1 A_1$ about the axis of moments is

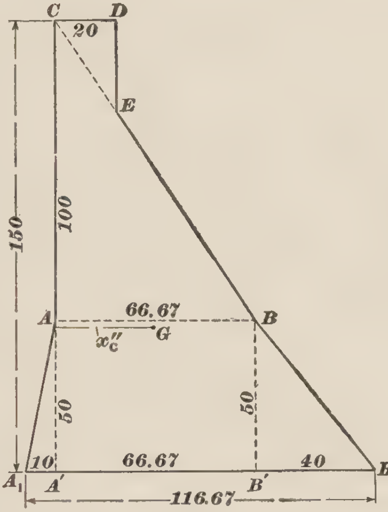


FIG. 20

$$3{,}333 \times 33.33 + 1{,}000 \times 80 - 250 \times 3.33 = 190{,}256$$

The area of the profile is $3{,}633 + 250 + 3{,}333 + 1{,}000 = 8{,}216$ square feet. The static moment of the part $A C D E B$ is 78,070. The distance x_c'' of the center of gravity of the entire area $A_1 A C D E B B_1$ from the axis of moments is, then, given by the equation

$$x_c'' = \frac{78{,}070 + 190{,}256}{8{,}216} = \frac{268{,}326}{8{,}216} = 32.66 \text{ feet}$$

The distance of the center of gravity from the toe A_1 is $32.66 + 10 = 42.66$ feet.

In Fig. 21, which shows the base $A_1 B_1$ of Fig. 20, the triangle of forces is constructed, the weight of the mass being $8{,}216 \times 140 = 1{,}150{,}240$ pounds, and the thrust of water, $31.25 \times 150^2 = 703{,}125$ pounds, applied at a height of $\frac{150}{3} = 50$ feet above the base $A_1 B_1$. To ascertain the maximum intensity of stress in the segment adjacent to A_1, when the reservoir is empty, we have

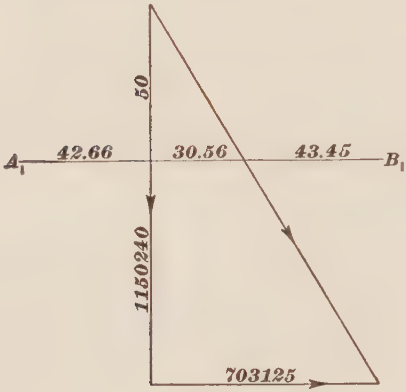


FIG. 21

$L = 116.67$, $d = 42.66$, $L - d = 74.01$, and $W = 1{,}150{,}240$. Substituting in formula **3** of Art. **35**,

$$P = \frac{1{,}150{,}240 \times 74.01}{116.67 \times 42.66} = 17{,}104 \text{ pounds}$$

When water pressure is added, $d = 43.45$, $L - d = 73.22$, and for the maximum intensity of stress on the segment adjacent to B_1, we have

$$P = \frac{1{,}150{,}240 \times 73.22}{116.67 \times 43.45} = 16{,}614 \text{ pounds}$$

This is a reasonable progression in the stresses, and so far shows a satisfactory profile.

40. For the next 50 feet, which will bring the profile up to the height of 200 feet, the base must widen more rapidly. A batter of 30 per cent. will be given to the back, and one of 100 per cent. to the face, as shown in Fig. 22. This adds to the base a triangle of 50 feet altitude and 15 feet base; and to the face, a rectangle of 50×116.67 and a triangle of 50 feet altitude and 50 feet base.

The axis of moments, which is $A' C$ in Fig. 20, is 10 feet to the right of $A_1 A''$, Fig. 22. The area of the

triangle A_1A_2A'' is $\frac{15 \times 50}{2} = 375$ square feet, and the center of gravity is $\frac{1}{3}(10 + 10 + 25) = 15$ feet to the left of the axis of moments. The area of the rectangle $A_1B_1A''B''$ is $50 \times 116.67 = 5{,}833.5$ square feet, and the center of gravity is $\frac{116.67}{2} - 10 = 48.33$ feet to the right of the axis of moments. The area of the triangle $B_1B''B_2$ is

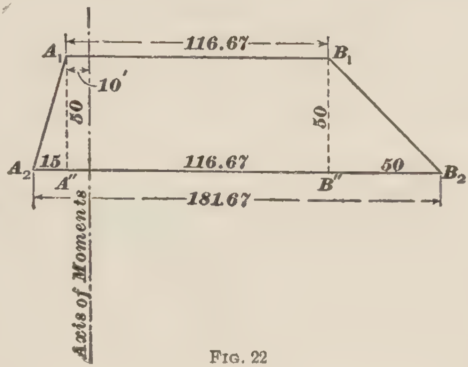


FIG. 22

$\frac{50 \times 50}{2} = 1{,}250$ square feet, and the center of gravity is $106.67 + \frac{50}{3} = 123.34$ feet to the right of the axis of moments. The static moment, about the axis of moments, of the area represented in Fig. 22, is, then,

$$5{,}833.5 \times 48.33 + 1{,}250 \times 123.34 - 375 \times 15 = 430{,}483$$

The area of the entire profile so far considered can now be found by addition:

$$8{,}216 + 375 + 5{,}833.5 + 1{,}250 = 15{,}674.5 \text{ square feet}$$

For the distance of the center of gravity of the entire profile from the axis of moments we have

$$x_c''' = \frac{78{,}070 + 190{,}256 + 430{,}483}{15{,}674.5} = 44.58 \text{ feet}$$

The distance of the center of gravity from the toe A_2 is $44.58 + 25 = 69.58$ feet.

In Fig. 23, which shows the base A_2B_2 of Fig. 22, the triangle of forces is constructed, the weight of the mass being $15{,}674.5 \times 140 = 2{,}194{,}430$ pounds, and the thrust of the water, $31.25 \times 200^2 = 1{,}250{,}000$ pounds, applied at a height of $\frac{200}{3}$ feet above the base.

To ascertain the maximum intensity of stress in the segment adjacent to A_2, with an empty reservoir, we have

$L = 181.67$, $d = 69.58$, $L - d = 112.09$, and $W = 2{,}194{,}430$. Substituting in formula **3** of Art. **35**,

$$P = \frac{2{,}194{,}430 \times 112.09}{181.67 \times 69.58} = 19{,}460 \text{ pounds}$$

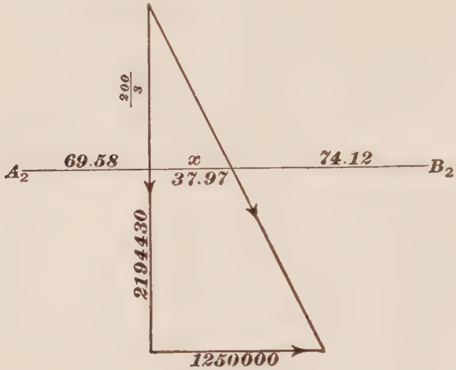


FIG. 23

and, with a full reservoir, on the segment adjacent to B_2,

$$P = \frac{2{,}194{,}430 \times 107.55}{181.67 \times 74.12} = 17{,}527 \text{ pounds}$$

This is also perfectly satisfactory under the given conditions.

41. The next 50 feet will complete the total height of 250 feet. The base is widened at the back, by adopting a batter of 60 per cent. On the face the limiting slope has been reached, so that the only means of widening out is by an offset. Accordingly, a step of 10 feet is made, from which the 45° slope is continued to the base. The additions are shown in Fig. 24.

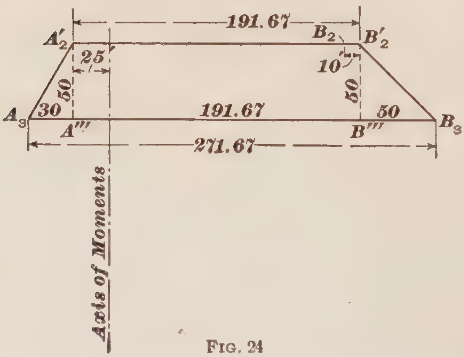


FIG. 24

The area of the triangle $A_3 A_2' A'''$ is $\frac{30 \times 50}{2} = 750$ square feet, and the center of gravity is $\frac{1}{3}(25 + 25 + 55) = 35$ feet to the left of the axis of moments. The area of the rectangle $A_2' B_2' B''' A'''$ is $50 \times 191.67 = 9{,}583.5$ square feet, and the center of gravity is $\frac{191.67}{2} - 25 = 70.83$ feet to the right of the axis of moments. The area of the triangle $B_2' B''' B_3$ is $\frac{50 \times 50}{2} = 1{,}250$ square feet, and the center of gravity is $\frac{1}{3}(166.67 + 166.67 + 216.67) = 183.33$ feet to the right of the axis of moments.

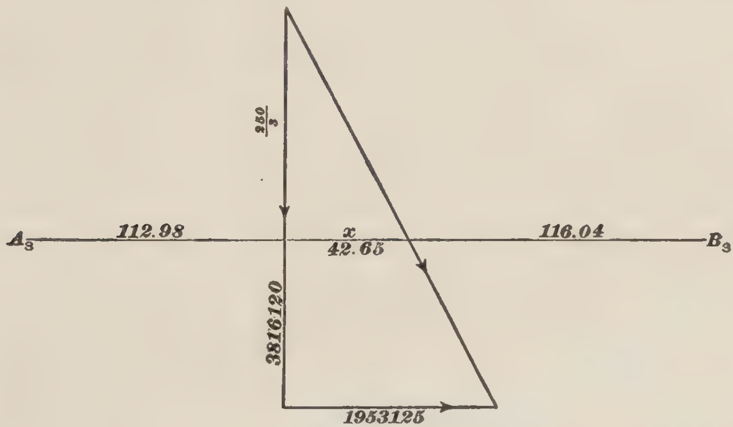


FIG. 25

The static moment of the area represented in Fig. 24 about the axis of moments is

$$9{,}583.5 \times 70.83 + 1{,}250 \times 183.33 - 750 \times 35 = 881{,}711.8$$

The area of the entire profile is now

$$15{,}674.5 + 750 + 9{,}583.5 + 1{,}250 = 27{,}258 \text{ square feet}$$

For the distance of the center of gravity of the entire profile from the axis of moments we have

$$x_c'''' = \frac{78{,}070 + 190{,}256 + 430{,}483 + 881{,}711.8}{27{,}258} = 57.98 \text{ feet}$$

The distance from the toe of the dam is $57.98 + 55 = 112.98$ feet.

All the data necessary for using formula **3** of Art. **35** are shown in Fig. 25. For the stress on the segment adjacent to A_3, $L = 271.67$, $d = 112.98$, $L - d = 158.69$, and $W = 3,816,120$. Substituting in the formula,

$$P = \frac{3,816,120 \times 158.69}{271.67 \times 112.98} = 19,730 \text{ pounds}$$

For the stress on the segment adjacent to B_3, $L = 271.67$, $d = 116.04$, $L - d = 155.63$, and $W = 3,816,120$. Substituting in the formula,

$$P = \frac{3,816,120 \times 155.63}{271.67 \times 116.04} = 18,839 \text{ pounds}$$

42. All the conditions of the problem have been complied with, so far as the superstructure is concerned. If, however, it had been desired to go another 50 feet higher, making the total height 300 feet, the base would have assumed extravagant dimensions. It would have been necessary to increase the batter on the back to 80 or 100 per cent. and to take another considerable offset on the face. It is evident, therefore, that, with the given conditions, 250 feet represents nearly the practical limiting height of a masonry dam.

43. Foundation Block.—It now remains to proportion the foundation block. It will be rectangular in section, and the limiting intensity of stress will be 30,000 pounds per square foot. The permissible intensity of stress is raised to this amount because the vertical sides are firmly and squarely compressed by the earth against them, and the masonry can, therefore, safely withstand a greater force. The two cases of a full and an empty reservoir are to be considered. The additional width must be given by front and back offsets.

In determining these offsets in the foundation, it will be best to make them so that the maximum intensity of stress will be the same for a full as for an empty reservoir. This end can be easily attained, since there is now no thrust of the water to be taken into consideration, and the addition consists of a rectangle. Fig. 25 shows that the short segment for an empty reservoir is 112.98 feet from the toe A_3, and 116.04 feet from the toe B_3 for a full reservoir. If an

offset of 14 feet is made at the back and one of 11 feet at the front, as shown in Fig. 26, the line of action of the weight of the dam, when the reservoir is empty, will meet the base MN of the foundation block at a distance of practically 127 feet from the end M, and the line of action of the vertical component of the pressure, when the reservoir is full, will cut MN at a distance of 127 feet from the end N. The width of the base MN is $271.67 + 14 + 11 = 296.67$; or, in round numbers, 297 feet. The maximum intensity of stress can now be determined for either a full or an empty reservoir; it is immaterial which condition is assumed, as the intensity of

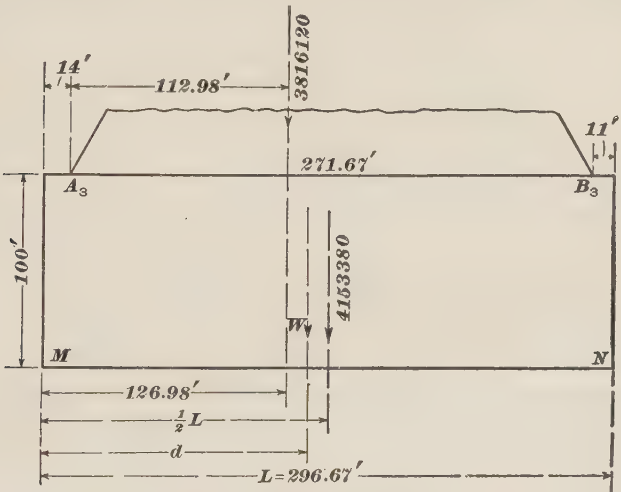


FIG. 26

stress is the same for both. Here an empty reservoir will be considered, and moments will be taken about M, in order to compute the distance d from the end M of the line of action of the total weight W resting on the base MN. The weight of the superstructure, as already determined, is 3,816,120 pounds, and its moment about M is $3{,}816{,}120 \times 126.98$. The weight of the foundation block, whose height will be made 100 feet, is $296.67 \times 100 \times 140$, and its moment about M is $296.67 \times 100 \times 140 \times \frac{296.67}{2}$. Therefore

$$d = \frac{3,816,120 \times 126.98 + 296.67 \times 100 \times 140 \times (296.67 \div 2)}{3,816,120 + 296.67 \times 100 \times 140}$$
$$= 138.11 \text{ feet}$$

To apply formula **3** of Art. **35**, the data are: $L = 296.67$,

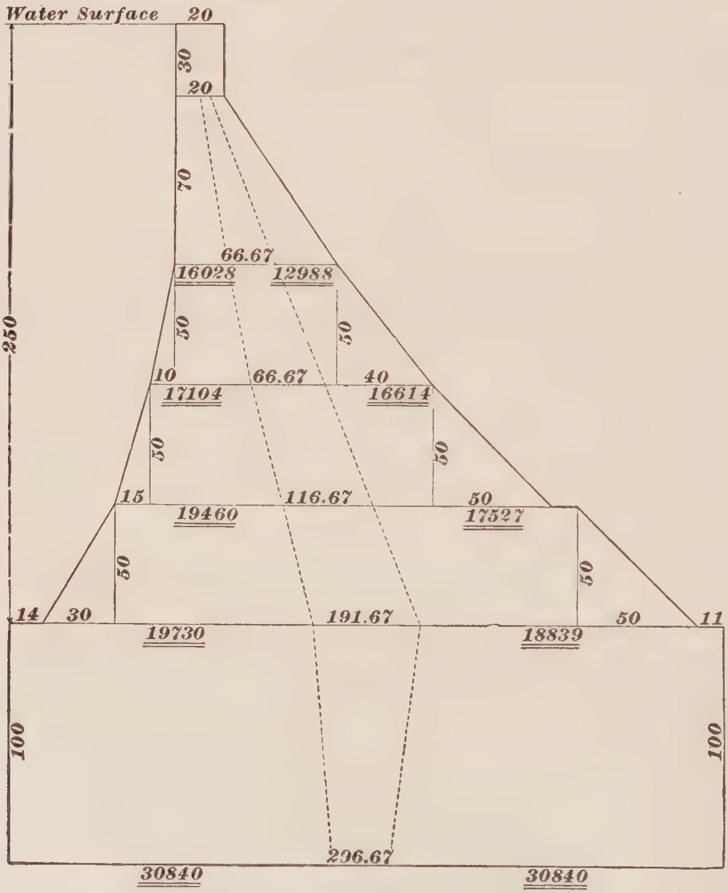


Fig. 27

$d = 138.11$, $L - d = 158.56$, and $W = 7,969,500$. Substituting in the formula,

$$P = \frac{7,969,500 \times 158.56}{296.67 \times 138.11} = 30,840 \text{ pounds}$$

This slightly exceeds the limit, but in a practical design it would be considered as fulfilling the conditions.

Fig. 27 shows the entire profile with maximum stresses in the corresponding segments, and in dotted lines the "curves of pressure" for an empty and a full reservoir. These curves are constructed by drawing a line through the points where the resultants cut the bases of the various partial profiles, as already determined, except for the upper 30 feet of the profile.

44. General Remarks.—The profile just established is typical, and applies rigorously to a structure weighing 140 pounds per cubic foot, with a top width of 20 feet. Neither the assumed weight per cubic foot nor the top width varies greatly from these values in practice; so that the general form of profile shown in Fig. 27 can be used for any height from 100 to 250 feet, although the dimensions may have to be slightly changed according to the data. This form will always furnish a safe basis from which to start, and each height can be tested with the new data precisely as has been done in the foregoing articles.

45. Accessories of High Dams.—For dams reaching and exceeding 100 feet in height, it is still more indispensable to provide a natural overflow or escape for surplus water than for lower dams. Generally, it is easier to find such an outlet for very high dams than for comparatively low ones, because the water-line approaches so nearly the surrounding summits. Indeed, in the case of high dams, it is frequently necessary to construct one or more auxiliary dams to prevent the water, when it rises to its full height, from escaping over the divides into other valleys. If it should become necessary, however, to allow the waste water to pass over the face of the dam or a portion of it, the profile already established will, in general, suffice for all heights where the base is equal to 90 per cent. of the height. When it falls below this, it should generally be brought up to this percentage by extending the outer toe B in the figures. No further general rule will be given, because all such

exceptional structures should be considered as special cases, and studied accordingly.

The means of controlling the water is the same in principle in masonry dams of all heights as for earthen dams. They are always simpler, however, because there is no earthen embankment, or at most a smaller one, to penetrate.

46. Execution of Work.—It is not sufficient that hydraulic work should be correctly designed; it is equally important that it should be carefully and properly executed. The closest attention to the most minute details is necessary. To indicate the manner in which the work should be carried on, the following extracts are given from Gould's *Specifications for Dams and Reservoirs*, which embody the specifications followed in the construction of the storage reservoirs of the Scranton Gas and Water Company, of Scranton, Pennsylvania:

(*a*) *Clearing and Grubbing.*—The whole area of the reservoir shall be cleared by cutting all trees, stumps, and bushes even with the ground and removing or burning the same. The area covered by the dam embankment shall be grubbed, so as to remove all stumps, roots, and sods.

(*b*) *Rock Excavation.*— * * * As it is important that, in the rock, the trenches shall be shattered as little as possible, hand drilling and light charges of explosives must alone be used.

(*c*) *Embankment.*—The material for the embankment shall be such as will produce a solid, water-tight bank, and shall be selected subject to the approval of the engineer. It must be taken as far as practicable from the land inside the reservoir. No stones larger than 2 inches in any direction will be allowed in the bank on the upper side of the center wall, or in any slopes exposed to the action of the water, and none larger than 4 inches on the lower side of said center wall. The material shall be laid down in horizontal courses, carried on the work in wagons, or carts, or in barrows, so as to insure its being well traveled over with a view to its consolidation. If, in the judgment of the engineer, these means do not produce a sufficiently compact bank, he may, in addition, order the use of rollers of a design approved by him, or such other means as may be necessary to secure the desired compactness. In all places that cannot otherwise be reached, the bank shall be tamped with heavy rammers. The bank shall be kept thoroughly moistened, while in process of construction, by means of pumps, hose, or sprinkling carts.

(*d*) *Rubble Masonry.*—The masonry for the center wall shall be composed of quarry stones containing not more than one-third of 1 cubic yard each, unless otherwise permitted by the engineer. They shall all have clean quarry faces, beds, and joints, and each stone shall be thoroughly wet just previous to being laid; every stone shall be laid in a full and swimming bed of mortar, and the interior vacancies shall be filled with mortar before any spalls or small stones are put in, the object being to make the wall perfectly water-tight. The bed and end joints of both faces of the wall shall be raked and struck as the work progresses. No stones will be allowed to be deposited nor dressed on the wall, but all stone shall be deposited and dressed on planks furnished by the contractor at his own expense, to prevent the stone from coming in contact with the dirt of the embankment. The center wall shall always be kept at least 2 feet above the adjacent embankment.

The above specifications apply generally to all rubble masonry to be built in this work, with the exception that, for work other than the center wall, larger stones will be permitted, and in some cases required. But all rubble masonry, especially where exposed to the action of the water, shall be strictly hydraulic, as described for the center wall.

The face of all rubble masonry, except the center wall, shall be hammer-dressed, and pitched to true and fair lines, with horizontal and vertical beds and joints, extending back at least 1 foot from the face. No spalls will be allowed in the face, nor any stones less than 3 inches thick. All work will be thoroughly bonded, with a proper proportion of headers, and breaking well the joints.

INTRODUCTION TO CONSTRUCTION DRAWING

GENERAL CONSIDERATIONS

DEFINITIONS

1. Mechanical Drawing.—The general geometrical principles that form the foundation of all instrumental drawing have been given and applied in *Geometrical Drawing.* The application of these principles to the representation of objects is usually called **mechanical drawing.** This term, however, is gradually being restricted so as to cover only the representation of objects connected with mechanical engineering. For this reason, it has been found necessary to apply special terms to the representation of objects in other lines of engineering. For example, the terms *architectural drawing* and *structural drawing* have come into common use to distinguish the branches of mechanical drawing that deal, respectively, with building and structural work. Architectural drawing consists of the representation of the general appearance, form, and design of a structure; few dimensions other than principal dimensions are given, the accuracy of the drawing being relied on to show the required sizes of parts. Structural drawing, on the other hand, deals with the structural features; all necessary dimensions are given, including the required sizes and arrangement of all parts. In addition, structural drawing usually includes the representation of all details.

2. Construction Drawing.—All drawings relating to civil-engineering construction come under mechanical drawing, taking the latter term in its broader sense. For convenience, however, drawing, when applied to civil-engineering work, will here be called **construction drawing.**

DRAWINGS

3. Classification of Drawings.—Drawings are usually classified, according to the purpose for which they are to be used, into *design drawings*, *detail drawings*, and *shop*, or *working*, *drawings*.

4. A **design drawing,** as the name implies, represents the general design, and gives the general arrangement and dimensions, of the finished work. This kind of drawing requires less work in preparation than any other, and is, therefore, not nearly so expensive. It is much used to show proposed work, sometimes as many as four or five designs being shown.

5. When the general design is adopted, more complete and elaborate drawings, called **detail drawings,** are made. They show all the information that is given on the adopted design drawing, and, in addition, the arrangement of all details. Sufficient dimensions and information are given on these drawings to compute all dimensions that are not directly given.

6. Detail drawings are often used by workmen in construction, but better results are obtained, and less trouble is encountered, if **shop, or working, drawings** are made. These drawings contain all the information usually given on design and detail drawings, and, in addition, all the actual dimensions of each separate piece that enters into the construction of the work. It is customary, in making working drawings, to make a general drawing showing the location of each part in the finished work, and also a separate drawing of each part, giving all the information that the workmen need for its construction. In making a working drawing,

the draftsman should keep in mind the fact that the drawing is to be used by workmen, whose knowledge of drawing is usually very limited; he should, therefore, endeavor to make it so plain that there will be no possibility of confusion or misunderstanding. The draftsman should consider the working drawings as a complete letter of instruction to the workman, telling him what material to use, what sizes to make the parts, and how to finish them. Working drawings should be so complete that it will not be necessary for workmen to lose time going to the drafting room for further information.

7. Paper Drawings.—Construction drawings are first made on paper in pencil, and the lines are not inked. For this purpose, a good grade of Manila paper on which lines can be easily drawn and erased should be used. A 4-h (HHHH) or 6-h (HHHHHH) pencil is well adapted to this kind of work. The lines should be made somewhat heavier than on drawings in which the lines are to be inked. These drawings are called **original**, or **pencil, drawings**, and are usually made complete, all information being given. They are sometimes filed away as records, but are usually destroyed after tracings have been made.

8. Tracings.—When the pencil drawings are finished, they are reproduced by tracing the lines on specially prepared cloth, called **tracing cloth.** Tracing cloth is linen treated with a waxy sizing so as to make it semitransparent. After it is sized, one side is dull and slightly rough, and the other is bright and smooth, presenting a glazed surface.

In copying a drawing on tracing cloth, the cloth is tacked over the drawing, and the lines on the cloth are drawn by following those on the drawing, which are plainly seen through the cloth. A drawing thus copied on tracing cloth is called a **tracing.** Tracings can also be made on **tracing paper,** which is transparent paper especially prepared for this purpose. When, however, a tracing is to have much handling, cloth is preferable to paper.

Some tracers prefer to draw on the dull, rough side, and others prefer the smooth side of tracing cloth. Many offices

adopt one side for all their draftsmen to use. The smooth side has the advantage that erasures are more easily made; however, it does not take ink so well as the rough side. When the smooth side is used, powdered chalk should be spread and rubbed over it; this removes the gloss from the surface of the cloth, which then takes in ink more easily. The student is advised to practice the use of both sides. For this purpose, he may draw in pencil some simple figures, such as triangles, rectangles, and circles, and make tracings, using sometimes one side of the cloth, sometimes the other.

9. Stretching Tracing Cloth.—In construction drawing, all shading is omitted, the surface or boundary lines being about the same thickness at every part of the drawing. Since tracing cloth is only semitransparent, the pencil lines on the paper drawing should be comparatively heavy, so that they will show through the tracing cloth when the latter is placed over them. In placing a sheet of tracing cloth over a pencil drawing, care should be taken that the pencil drawing is stretched tight and smooth on the drawing board, and that the tracing cloth is so stretched and tacked down on top of the paper drawing that there is no unevenness. In case there are any wrinkles in the cloth after it is laid, they can be removed by taking out one or two tacks at a time and smoothing the cloth outwards with the palm of the hand, replacing each tack as a part of the cloth is smoothed.

BLUEPRINTING

10. Blueprints.—When tracings are finished, as many copies of them as are wanted are obtained by a special process called **blueprinting.** These copies are called **blueprints,** and are taken on **sensitized paper.** This paper is coated on one side with a chemical preparation that can be removed at any time, if it has not been exposed to bright daylight or sunlight, by immersing the paper in water; after a short washing, the coating is entirely removed and the paper resumes its original color. If the coating is exposed to bright daylight or sunlight, it undergoes a chemical

change that colors the surface of the paper, and it is then impossible to remove the color by washing. This principle is made use of in blueprinting. The tracing is placed on top of a sheet of sensitized paper in such a way that the coated surface of the paper, and the ink lines on the tracing, are uppermost. A sheet of glass is then placed on top of the tracing, and the whole is exposed to the brightest available light. That part of the coating on the sensitized paper that is directly under the ink lines on the tracing is protected from the light, and undergoes no change; all other parts undergo the change referred to, the color of the surface of the paper being changed. When the drawing has been exposed for a sufficient length of time, it is removed from the light; the sensitized paper is at once immersed in a large basin or sink containing clear water, and thoroughly washed. The coating is washed off all that part of the paper that was covered by the ink lines, leaving the paper white; all the remainder of the paper is colored. The result is an exact reproduction of the tracing. As many prints as desired can be made, one at a time, from a tracing.

11. Sensitized Paper.—Sensitized paper can be purchased in rolls of almost any desired width and length all ready for use. There are several kinds on the market, which differ only in the color of the finished print and in the method of washing they require: the one most used and most economical is *blueprint paper*, so called because all parts of the print, except the lines, notes, and dimensions are blue. Blueprint paper should be kept in a closed tin tube or in a drawer in a darkened room. It should under no circumstances be handled in the full light of day until after it has been washed.

12. Directions for Making Blueprint Paper.—It is sometimes desired to prepare instead of buying the sensitized paper. This is usually done as follows: Dissolve 2 ounces of citrate of iron and ammonia in 8 ounces of water; also, $1\frac{1}{4}$ ounces of red prussiate of potash in 8 ounces of water. Keep the solutions separate and in dark-colored bottles in a dark place. Better results will be

obtained if $\frac{1}{2}$ ounce of gum arabic is added to each solution. When ready to prepare the paper, mix equal portions of the two solutions, and be particularly careful not to allow any more light to strike the mixture than is absolutely needed. For this reason, it is necessary to have a dark room in which to work. There must be in this room a tray or sink of some kind that will hold water; it should be larger than the blueprint and about 6 inches deep. There should also be a flat board large enough to cover the tray or sink. There must be an arrangement like a towel rack to hang the prints on while they are drying; this arrangement will be called a **print rack.** The paper used for blueprinting should be a good, smooth, white paper, and may be purchased of any dealer in drawing materials. Cut it into sheets a little larger than the tracing, so as to leave an edge around it when the tracing is placed on it. Place eight or ten of these sheets on the flat board before mentioned, taking care to spread them flatly one above another, so that the edges do not overlap. Secure the sheets to the board by driving a brad or small wire nail through the two upper corners sufficiently far into the board to hold the weight of the papers when the board is placed in a vertical position. Lay the board on the edges of the sink, so that one edge is against the wall and the board is inclined at an angle of about 60° with the horizontal. Darken the room as much as possible, and obtain whatever light may be necessary from a lamp or gas jet, which should be turned down very low. With a wide camel's-hair brush or a fine sponge, spread the solution just prepared over the top sheet of paper. Be sure to cover every spot, and do not get too much on the paper. Distribute the solution as evenly as possible over the paper, in much the same manner that the finishing coat of varnish would be put on by a painter. Remove the sheet by pulling on the lower edge, tearing it from the nail that holds it, and place it in a drawer where it can lie flat and be kept from the light. Treat the next sheet and each succeeding sheet in exactly the same manner, until the required number of sheets have been prepared.

Unless a large number of prints are constantly used, it is cheaper to buy the paper already prepared. There is scarcely anything saved in preparing the paper, and better results are usually obtained from the commercial sensitized paper, since the manufacturers have machines for applying the solution, and are able to distribute it very evenly.

13. Directions for Making Blueprints.—In Figs. 1 and 2 are shown two views of a printing frame that is well adapted to sheets not over 17 in. × 21 in. The frame is

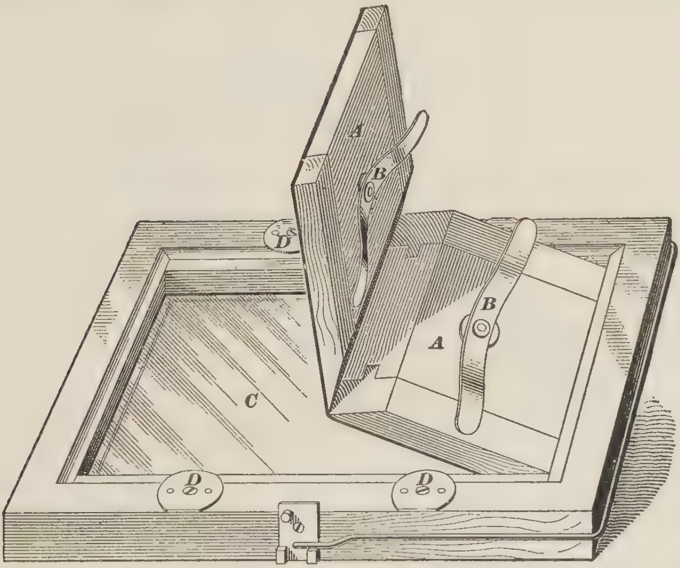


FIG. 1

placed face downwards, and the back *A* is removed by unhooking the brass spring clips *B*, *B*, and lifting it out. The tracing is laid on the glass *C*, with the inked side touching the glass. A sheet of the prepared paper, perfectly dry, is laid on the tracing with the sensitized side downwards. The paper and tracing are smoothed out so that they will lie perfectly flat on the glass, the cover *A* is replaced, and the brass spring clips *B*, *B* are sprung under the plates *D*, so that the back cannot fall out. While all this is being done,

the paper should be kept from the light as much as possible. The frame is now placed where the sun can shine on it, and is adjusted, as shown in Fig. 2, so that it will get the most light, or, if the sun is shining, so that the sun's rays will fall on it as nearly at right angles as possible. According to the conditions of the sky—whether clear or cloudy—and the time of the year, the print must be exposed from 3 to 15 minutes. The tray, or sink, already mentioned should be filled to a depth of about 2 inches with clear water (rain water if possible). The print having been exposed the proper length of

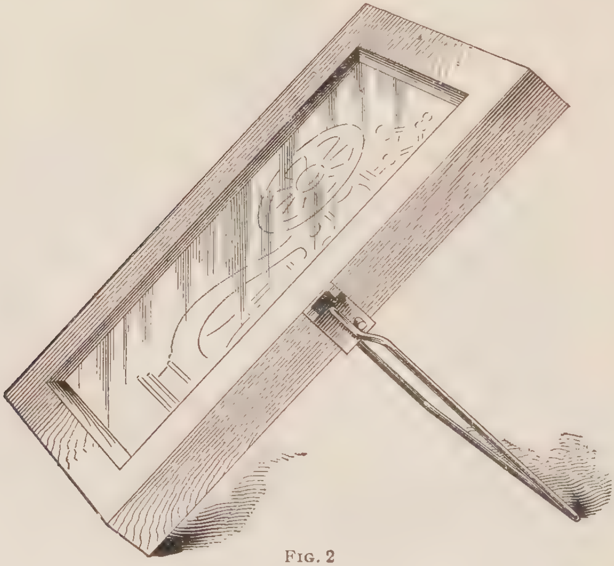


FIG. 2

time, the frame is carried into a dark part of the room, or the curtains at the windows are drawn, and the print is taken out of the frame. The print is then placed in the water with the sensitized side down, care being taken that the paper lies flat and that there are no air bubbles under it. The print is allowed to soak for from 3 to 6 minutes, ordinarily while the next print is being put in the frame. The print is then turned over in the water and laid down flat with the blue side up. Water is then splashed over it with the hands so as

to thoroughly rinse the chemicals from the surface. The print is then taken from the sink and suspended by its upper edge to dry on the print rack. The proper duration of the bath can best be determined by observation; dark purple- or bronze-colored spots on the prints indicate that the prints were not washed thoroughly on those spots. The prints should be again immersed in the water and washed. There is very little danger that the bath will be too long.

14. Time of Exposure.—The most important point in connection with blueprinting is the time of exposure. If a print is exposed for too short a time, the background or blue portion is very pale, and it is difficult to distinguish the lines and letters. If the print is exposed for too long a time, the background becomes dark blue or purple, and the light penetrates through the ink lines, thereby coloring the paper where it should remain unaffected. The time of exposure varies with the weather conditions, being less on a bright sunshiny day than on a dark cloudy day. On a cloudy day, when the sun is alternately visible and obscured, it may take several times as long for one print as for another. It is best to judge the proper time of exposure to the light by the color of the strip of print projecting beyond the edge of the tracing. To obtain the exact shade of the projecting edge, take a strip of blueprint paper about 12 or 15 inches long and 3 or 4 inches wide. Divide it into twelve or fifteen equal parts and mark them on the back 1, 2, 3, etc. Take a piece of tracing cloth, and place the blueprint paper and cloth in the frame in such a way that the cloth covers one-half the width of the paper. Now expose the paper to the light, and at the end of 1 minute cover the part of the strip marked 1 with a thin board or anything that will prevent the light from striking the part covered. At the end of the second minute, cover parts 2 and 1; at the end of the third minute, parts 3, 2, and 1; etc. Part 1 will have been exposed 1 minute; part 2, 2 minutes; part 3, 3 minutes; etc. When the whole strip is covered, remove the blueprint paper, cut the part that was under the tracing cloth from the remainder, and wash the

former thoroughly. When it has dried, select a good rich shade of blue and notice the number of this section on the back. Now observe carefully the color of the same portion

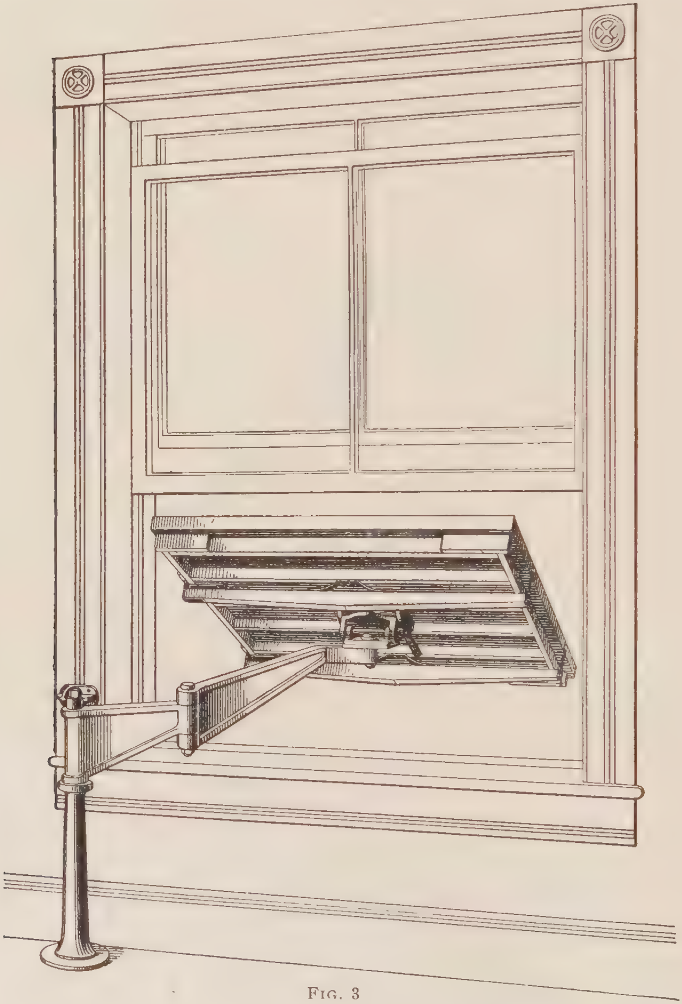


Fig. 3

of the remainder of the strip; this is the desired color of the protruding edge and should be kept in mind. All prints should be exposed until the protruding edge attains this color.

15. Large Frames.—As stated in Art. **13**, the type of frame shown in Fig. 1 is well adapted to small-sized drawings. For large drawings, the same general type of frame is employed, but it is usually supported by some device. In Fig. 3 is shown a patented frame that can be placed outside the window and adjusted to any angle. When not in use, it can be folded up against the wall. It is made in different sizes, from 16 in. × 24 in. to 48 in. × 72 in.

16. Rolling Frames.—The larger sizes of blueprint frames are too heavy for convenient handling, and it is

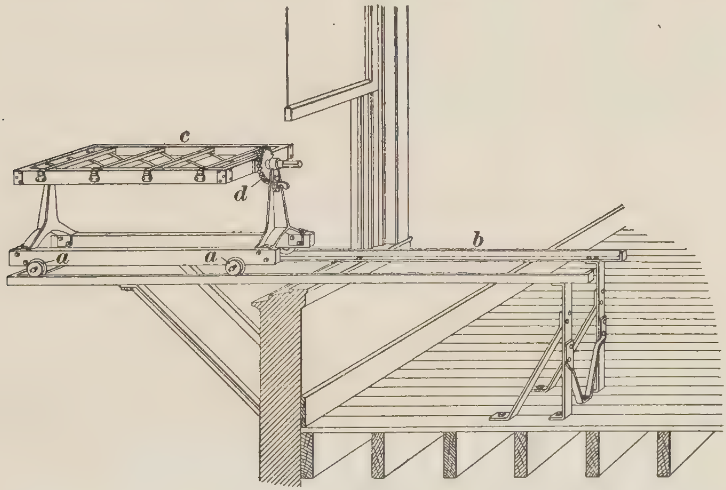


FIG. 4

customary to mount them on a support, as shown in Fig. 4. The support is mounted on rollers *a*, *a*, and runs on a track *b* that extends through a window opening into the outer air. The frame *c* is shown bottom up in the figure, having been run out this way. It can be turned to any angle by hand and held at that angle by means of the pawl and ratchet *d*. This form of frame and support is very serviceable.

17. Electric-Light Process.—Recently, an apparatus has been perfected for taking blueprints by electric light. The device generally consists of a glass cylinder properly

mounted and provided with a canvas cover, or sheet, that is wrapped, or drawn, around the outside, and in this way holds the tracing and sensitized paper against the glass. When set in this manner, an arc light is slowly lowered inside the cylinder. When the light reaches the bottom of the cylinder, the print is taken out and washed. The time of exposure of the print is regulated by the speed of the descending arc, which is controlled by some mechanical device.

18. Blueprint Cloth.—Blueprints are frequently made on cloth instead of on paper. The process is the same as for printing on paper. Cloth blueprints are more expensive than paper blueprints, but for working drawings that will be much handled by the workmen, they are better, as they stand wear and tear much better than paper prints.

DRAFTSMEN

19. Grades of Draftsmen.—In a well-equipped engineering office, where a large number of draftsmen are employed, it is customary to divide them, according to their work, into several classes. The usual classes are *designers*, *detailers*, *tracers*, and *checkers*.

20. Designers are the most proficient, and although many of them do very little drafting, they are classed under the head of draftsmen. They design the work and compute the strength and required sizes of all parts. In some drafting rooms, particularly when an unusual piece of work is under way, the designer makes numerous rough sketches illustrating the general arrangement of parts, and gives them to the detailer to aid him in the preparation of the drawings. In addition to having a knowledge of mechanical drafting, the designer must be familiar with the engineering principles involved in the construction of the work he designs.

21. Detailers prepare the design, detail, or working drawings, as the case may be, from information and instructions given by the designers. They compute all secondary

dimensions, such as rivet spacing, thickness of retaining walls and dams at different heights, etc. from the principal dimensions furnished by the designer. Detailers should have a thorough knowledge of mechanical drafting, and should be well acquainted with the standard details of the types of construction in which they are engaged.

22. Head Draftsman.—In almost all drafting rooms there is a head, or chief, draftsman, who supervises the work of the detailers and tracers. He receives general information and instructions from the designers, which he transmits to the detailers, adding small details and special instructions to expedite the making of the drawings. He gives to each detailer, as far as possible, the class of work to which the detailer is best adapted, either from natural aptitude or from previous experience.

23. Tracers.—Detailers usually make their drawings in pencil on paper. The drawings are then turned over to less experienced men to be traced. These men are called **tracers**. Their knowledge need not extend beyond familiarity with the use of instruments. Tracers should be neat, accurate, and rapid workers, and fairly good letterers. A draftsman in this position has very good opportunities to become acquainted with the best types of construction, and, if he has an aptitude for drawing and some knowledge of engineering principles, can advance very rapidly

24. Checkers.—When tracings are completed, they are turned over to a man to be carefully examined for mistakes. This man is called a **checker**, and usually examines the work of the designer, detailer, and tracer. When the drawings are checked, the checker places his initials on them, and thereafter he is responsible for their correctness. The checker is usually a man that has had experience as a tracer, detailer, and designer, and has demonstrated that he can be relied on for accuracy and faithfulness. In spite of the utmost care, designers, draftsmen, and tracers make mistakes, and it is the duty of the checker to find them. He should never assume or take for granted that anything on

the drawings is correct. The checker marks on the tracings whatever mistakes he finds, and then hands the tracings to the tracers for correction.

25. Small Drafting Forces.—In many offices where few draftsmen are employed, the positions outlined in the preceding articles are frequently combined. For example, designers are sometimes also detailers, and may trace their own drawings. More seldom, draftsmen may be found that design, draw, trace, and check their own work. This course is very objectionable: to avoid mistakes, at least two men should look over each drawing.

26. Blueprint Clerk.—A position that does not strictly come under the head of draftsman, but is closely allied to it, is that of **blueprint clerk**, or **blueprint boy**. In all large engineering offices, blueprints are made by one man, in order to avoid confusion and loss of time. It is customary to turn the tracings over to the blueprint clerk with a note as to the number of prints required. He returns the tracing with the required number of prints when they are dry. Almost any one can become proficient in blueprinting in less than 1 day, no special talent being required—nothing in fact but a little common sense. The position is mentioned here because the blueprint clerk has a good opportunity to become acquainted with different types of construction. If he is attentive and ambitious, and has some knowledge of drawing, he will soon know how to read plans and may secure a position as tracer.

SCALES

27. Scale of a Drawing.—It is seldom convenient to make full size the drawing of an object. The ratio of the dimensions on a drawing to the corresponding dimensions of the object represented is called the **scale** of the drawing. Thus, if every line in the drawing is one-fourth of the corresponding line on the object, the scale of the drawing is $\frac{1}{4}$, or 1 to 4. Since, in such a case, every foot on the object is

represented by 3 inches on the drawing, this scale is also expressed as **3 inches to the foot.** Similarly, a scale of $1\frac{1}{2}$ inches to the foot is one in which a distance of 1 foot on the object is represented on the drawing by $1\frac{1}{2}$ inches. This scale is also expressed as a $\frac{1}{8}$ scale, since the ratio of any line on the drawing to the corresponding line on the object is $\frac{1}{8}$.

In some cases, the drawing is made to a scale greater than 1; that is, the lines on the drawing are larger than the corresponding lines on the object. This is usually the case in the representation of very small objects or details.

The dimensions given on a drawing are always followed; but it is customary to state the scale to which drawings are made, as this adds to the clearness of the drawing. It is common practice to have parts on the same sheet drawn to different scales, in order to make every part sufficiently clear.

28. Measuring Scales.—The name *scale* is also given to a graduated rule used for measuring distances. Some of these rules are graduated to a **natural scale**, that is, into true inches and fractions of an inch, such as eighths, sixteenths, tenths, etc. Others are graduated in such a manner as to give directly, without computation, the proper reduced lengths of lines drawn to the scale adopted for the drawing.

Fig. 5 shows a convenient scale, which combines eleven systems of subdivision, and may be used for all the mechanical drawing ordinarily done in a drafting room. This scale is triangular in section, and its graduations cover a length of 12 inches. On each of its edges is laid off a scale, as shown at A, B, and G. The scale at G is **full size**, or natural; this edge of the scale is divided into inches and fractions of an inch down to sixteenths, and is used for drawings in which an object is represented in its natural size. On the opposite side, at B, is shown the quarter-size scale of 3 inches to the foot. This is indicated by the *3* at the end. The first 3-inch (actual size) division from B to C is subdivided into twelve parts representing inches, and each of these parts is then subdivided into equal parts, generally to represent fractions of an inch. From C to D, D to E, and E to F, the scale is marked in its

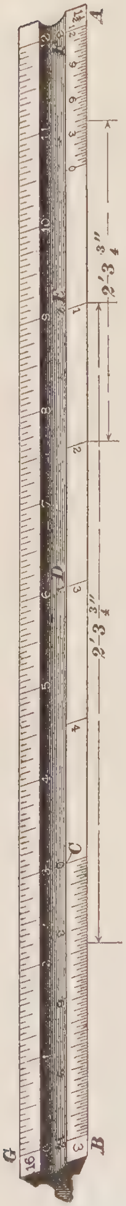


FIG. 5

main divisions to represent 1 foot each, each foot being represented on the scale by an actual length of 3 inches. From *A* to *B*, the scale is independently divided into spaces of $1\frac{1}{2}$ inches (actual size) to form an eighth-size scale, or $1\frac{1}{2}$ inches to the foot; this is indicated by the $1\frac{1}{2}$ at the end. The divisions of the latter scale occur on and between the marks for the 3-inches-to-the-foot scale.

The other sides and edges of the instrument are divided into scales of 1 and $\frac{1}{2}$, $\frac{3}{4}$ and $\frac{3}{8}$, $\frac{1}{4}$ and $\frac{1}{8}$, and $\frac{3}{16}$ and $\frac{3}{32}$ inch to the foot, each scale being designated by the fraction at its end. Different makers do not always arrange their scales in the same manner. Thus, instead of having a full-size scale and scales of 3 inches to the foot and $1\frac{1}{2}$ inches to the foot on one side, as shown in Fig. 5, some makers have the full-size scale, and the scale of $\frac{3}{16}$ and $\frac{3}{32}$ inch to the foot on one side. It will be noticed that the numbering of the feet on these scales does not start at the end of the instrument, but at the first main division from the end. Thus, on the quarter-size scale, or scale of 3 inches to the foot, the zero mark is placed at *C*, and the first foot is measured to *D*. This is done so that the feet and inches may be laid off independently and with one reading of the scale.

The figures indicating the number of feet on this scale are placed along the inside edge at *D*, *E*, and *F*, the numbers running in a direction away from the part containing the inches. The numbers indicating inches run in an opposite direction from those defining the feet.

29. Laying Off Distances to Scale.—To lay off 2 feet $3\frac{3}{4}$ inches on a scale of 3 inches to the foot, and from a given point, place the

scale on the point so that the 2-foot mark will be directly over it; then from the zero mark *C* lay off $3\frac{3}{4}$ inches as shown, locating a second point. The length of the distance thus laid off between the two points represents 2 feet $3\frac{3}{4}$ inches to the scale of 3 inches to the foot. The scale of $1\frac{1}{2}$ inches to the foot is used in a similar manner to lay off the same distance. The figures indicating feet on this scale are placed nearer the edge, in order to prevent confusion in reading.

To draw to half size, or to a scale of 6 inches to the foot, use the full-size scale, and remember that every $\frac{1}{2}$ inch on that scale corresponds to 1 inch on the object—that is, that every dimension on the drawing is only one-half of the real length. Thus, to lay off $5\frac{7}{8}$ inches, lay off 5 half inches and $\frac{7}{16}$ inch over; the result represents a line $5\frac{7}{8}$ inches long to a scale of 6 inches to the foot.

30. Special Scales.—It may happen that a draftsman is obliged to make a scale, when the size of his plate is limited and a general drawing of some object is desired. In such a case, one scale may be too large for the drawing to be made on a sheet of the required size; another scale may make the drawing too small. For example, a $\frac{1}{8}$ scale may be too large and a $\frac{1}{16}$ scale too small; a $\frac{1}{10}$ scale may be

FIG. 6

just right. If the draftsman has no $\frac{1}{10}$ scale, he may make one by taking a piece of heavy drawing paper and cutting out a strip about the size of an ordinary scale and laying off 1 foot on it, dividing it into ten equal parts. Each division or part will represent 1 foot on the object. Divide one of the end parts into twelve equal parts, and each will represent 1 inch on the object. Lines indicating half and quarter inches may be drawn if considered necessary.

Fig. 6 shows part of a scale made in this manner, giving feet, inches, and half inches—the quarters, eighths, etc. of an inch being judged by the eye.

To make a $\frac{1}{5}$ scale, lay off 12 inches and divide this distance into five equal parts. Divide one of the end divisions into twelve equal parts, to represent inches, and then divide each of these parts into halves, quarters, eighths, etc., as far as desired.

DIRECTIONS FOR DRAWING THE PLATES

GENERAL INSTRUCTIONS

NOTE.—The general instructions that follow apply to all the Drawing Plates given in the present and in succeeding Sections.

31. Size of Plates.—The plates are to be of the same size as those drawn in connection with *Geometrical Drawing;* that is, 14 in. $\times$ 18 in., with border lines drawn $\frac{1}{2}$ inch from the edge, making the working space 13 in. $\times$ 17 in.

32. Title and Number of Plate.—The title or name of each plate is, when convenient, to be placed at the lower right corner of the sheet; a space of $1\frac{1}{2}$ in. $\times$ 4 in. is to be reserved for this purpose. In addition to the title, there will be given in this space the number of the drawing, the draftsman's name, and the date when the drawing was finished. If desired, the date when the drawing was started may also be given.

33. Abbreviations.—In general, abbreviations are used on drawings only when lack of space prevents the use of complete words. There are, however, a few abbreviations that can be used without hesitation, having been fixed by long practice, such as D. or "Diam." for diameter; R. or "Rad." for radius; "Thds." for threads; f. for finish; C. L. or ℄ for center line.

34. Dimensions.—As a rule, all dimensions above 12 inches are to be given in feet and inches; the feet being

designated by a single accent mark (′), and the inches by a double accent mark (″). The feet and inches should be separated by a short dash; thus, 2′–3″.

Fractions should be written with the dividing line between the numerator and the denominator in a horizontal position, thus: $\frac{3}{4}''$, $\frac{7}{16}''$; never with the dividing line vertical or inclined, thus 3|4″, 7/16″. Mixed numbers should be written without a dash between the whole number and the fraction; thus: $4\frac{1}{2}''$, $5\frac{3}{4}''$. In writing decimals less than 1, a zero should generally be placed before the decimal point to avoid confusion; thus, 0.15″, 0.25″, not .15″, .25″.

Where possible, dimensions should be written so that they can be read from the bottom and the right side of the drawing.

35. Notes on Drawings.—When it is desired to convey special information, this is done by writing notes on the drawings. The location of notes should be chosen with great care, so that they will not be obscured by the lines of the drawing. The notes should be as clear and concise as it is possible to make them without omitting necessary information.

36. Drawing the Plates.—The method of making the drawings in this and succeeding Sections is different from that followed in *Geometrical Drawing*. The plates will first be made in pencil on Manila paper, but will not be inked in. The pencil lines should be about 50 per cent. heavier than those in the *Geometrical Drawing* plates. This is to render them more clearly visible through the tracing cloth (see Art. **9**). All lettering should be done on the pencil drawing before tracing, so that it will be properly and conveniently arranged.

37. Making the Tracings.—When the student finishes a plate in pencil, he should compare it carefully with the one sent by the Schools, and read again the directions given in the text. When he is certain that his plate has been drawn correctly, he should spread the tracing cloth over it, and stretch it smooth and flat, as described in Art. **9.** If the

smooth side of the cloth is used, it should be thoroughly rubbed with powdered chalk, magnesia, or pumice stone (any of which can be obtained at a drug store for a few cents), in order to remove all grease and dirt, and to slightly lessen the gloss on the surface, so the ink will flow evenly. When the entire surface has been rubbed, the remaining powder can be dusted off with a clean handkerchief or other cloth. Be careful not to use a dirty cloth, especially one that is likely to contain grease. Next fill the right-line pen and adjust it to the proper width of the line, trying it on the edge of the tracing cloth outside of the boundary lines, until it is right and the ink flows smoothly. Now, *without delay*, start drawing the lines on the tracing. Any delay allows the ink to dry and harden at the point of the pen, and this makes it necessary to clean out the pen and refill it. It is best to begin with the horizontal lines, then ink the vertical lines, and finally the inclined lines. When there are curves tangent to straight lines, the curves should be traced first. In this way, a better junction can be made than if the straight lines are drawn first. It should be remembered that ink dries slowly on tracing cloth, so that care must be taken not to smudge lines by rubbing them before they are dry. *Lines must be allowed to dry by themselves; they should not be dried with blotting paper*, as this paper absorbs part of the ink and leaves a pale line that does not show well on the blueprint. After a little experience, the draftsman will learn to divide his work in such a manner that he can work continually, moving from one part of the drawing to another when the lines are dry. Lettering, dimensions, and titles should not be inked in until all the lines are drawn.

38. Method of Handling the Ruling Pen.—In drawing a line, the pen should not press on the cloth very hard, and should move with almost uniform speed, as this gives a smoother line. Care should be taken to start a line at the right point, and not to carry it beyond nor stop it short of the end. At first, it may be found somewhat difficult to get lines that are smooth, distinct, and uniformly black; but,

after a little practice, this difficulty will disappear. It should be kept in mind that, in order to obtain a good line, it is not necessary to press the pen hard on the cloth. Ink work is entirely different from pencil work in this respect. It is sufficient to press the pen lightly, so that it will be well in contact with the surface of the cloth, to make the ink flow smoothly. An even pressure is necessary, however; good lines cannot be obtained if the pen just touches at one spot and digs in another. Never have the nibs of the pen in contact, even for the finest lines, as smooth lines cannot then be drawn. After a little practice, the tracer learns to get a fine line by spacing the nibs a very small distance apart. Each time the pen is filled, the width of line should be tried on the edge of the sheet of tracing cloth to see if the nibs need to be readjusted.

When it is necessary to draw pencil lines on a tracing, they should be made with a soft pencil, preferably softer than HH, as hard pencils cut into the cloth and the lines are difficult to erase.

39. Erasing.—In case it is necessary to erase an ink line from a tracing, this can be done by means of an ink eraser. Do not try to remove the line quickly by rubbing hard, but rub gently until the line is removed. Then rub some chalk on the surface where the erasure has been made. If the ink runs through the cloth, wait until the tracing is finished, then turn it over and remove the ink by means of the ink eraser. Have a blotter constantly at hand, so that if a blot occurs it can be removed at once. Do not lay the blotter flat on the paper on top of a blot, but first dip the corner of the blotter into the ink and allow it to absorb all the ink it will. Then place the blotter on top of the spot, and press it down gently. What remains of the blot after this can be entirely removed by means of the ink eraser, care being taken to allow the blot to dry entirely before the eraser is used.

40. Erasers.—The kind of rubber to be used varies somewhat with the nature of the material on which the drawing is made If bristol board, tracing cloth, or paper

with a coarse grain or fiber, which is apt to raise by rubbing, is used, a soft rubber is recommended. On duplex, double elephant, and the ordinary drawing papers, a harder rubber will be found better.

For erasing ink lines, ink erasers are most satisfactory. They are made with emery or glass mixed with the rubber; the action of the eraser cuts the outer skin of the paper on which the lines are drawn and removes the ink. If there should be two or three erasures over the same spot, the paper or cloth is very apt to rub through. With double elephant or any good drawing paper, this is not so likely to occur with two or three erasures; but when a line is to be erased from tracing cloth care must be exercised, because the cloth will not stand much rubbing without wearing through.

Before attempting to remove a line, the eraser should be cleaned by rubbing it on a clean piece of paper. Should it then smear the ink, it is probable that it is too hard or old, and it should be discarded. A rubber of the size and shape shown in Fig. 7 is usually found to be most convenient.

FIG. 7

When erasing on tracing cloth, the cloth should be laid on a smooth surface and held firmly, the rubbing being done gently and patiently until all traces of the lines have disappeared. A rotary motion imparted to the eraser often facilitates the operation. The gloss of the tracing cloth, which may have been destroyed by rubbing, can be renewed by applying powdered pumice stone with a chamois skin and finishing the spot with chamois alone. In erasing ink lines from drawing paper, the gloss may be brought back by burnishing the spot with a smooth piece of ivory, such as is used for the handles of drawing pens. By this means, a smooth surface is obtained on which the ink will not run.

41. Erasing Shields.—In Fig. 8 is shown a brass shield that is often found useful when erasing small spots,

figures, or short lines. It contains holes of different shapes, so that it is possible to erase particular spots without injuring adjacent parts of the drawing. As a soiled brass shield is likely to smut the drawing, care should be taken that the shield is perfectly clean. Shields made from thick drawing paper or thin cardboard will not soil the drawing, and have the additional advantage that they may be cut to the exact size or shape of the part to be erased; on the other hand, they have the disadvantage of wearing away sooner than those made of brass.

FIG. 8

42. Cleaning Drawings.—A drawing is almost sure to become soiled from the rubbing of the draftsman's sleeves, the perspiration from his hands, and from dust that accumulates on it. This may be prevented to a certain extent by covering the work, except those parts on which the draftsman is engaged, with transparent paper thoroughly secured at the edges so as not to interfere with the operation of the triangles and T square. After a drawing has been worked on for some time, an erasure brings out the original color of the paper, showing the amount of dirt that has accumulated.

Before commencing work, the scales, triangles, and T square should be thoroughly cleaned. One of the main sources of dirt on a drawing is the sliding of the instruments over the surface; celluloid and rubber triangles are especially liable to accumulate dirt. Any particles of pencil chips or rubber that may accumulate underneath the paper will raise small hills on the surface; consequently, the drawing board should be thoroughly cleaned before the paper is tacked in place. For cleaning drawings, sponge rubbers are by far the most effectual means. Though a piece of stale, well-kneaded bread will clean a drawing, or at least will distribute the dirt so that the drawing will have a cleanly appearance, it is not recommended.

43. Cleaning Tracings.—Tracings can be cleaned by means of a soft cloth soaked in benzine. This effectually

removes all dirt and pencil marks without injuring either the tracing cloth or the ink lines. Care should be taken, where it is desirable to clean the tracing in this way, that the ink is a good waterproof India ink. It is always advisable to experiment with the ink on a small piece of cloth before using it on the drawing. The benzine should be pure and contain no water, as water will destroy the tracing cloth and smear the ink.

44. Lettering.—Great stress was laid in *Geometrical Drawing* on the necessity of learning how to letter neatly. The student should remember that facility in lettering is acquired only by practice, and he should not become discouraged if his lettering on the first plates is not so good as that on the Schools' plates. To further assist him in becoming acquainted with the proper method of lettering the drawings, some additional information on lettering is here given.

The lettering on the plates in *Geometrical Drawing* is frequently used on mechanical and engineering drawings. The type used for the title is known as the *block letter*, while the single-line Italic is used for all notes and explanations. The lettering of the titles on these plates, while very legible and presenting a good appearance, cannot always be executed as quickly as the exigencies of the drafting room require; therefore, some other style of lettering is usually adopted.

45. Vertical Block Letters.—The block letter illustrated and described in *Geometrical Drawing* is sometimes modified by making the corners angular instead of round, as shown in Fig. 9. All these letters can be made with the right-line drawing pen, and when carefully laid out present a very good appearance. The total width of each letter, except *A*, *G*, *I*, *L*, *M*, *W*, *Y*, and the figures *1* and *4*, is four-fifths the height. The thickness or width of *1* and *I* depends on the size of the line used, the size or thickness of line in the figure being one-fifth the height. The width of *L* is usually made a little less than four-fifths the height. The total widths of *A*, *M*, and *Y* are each made equal to the

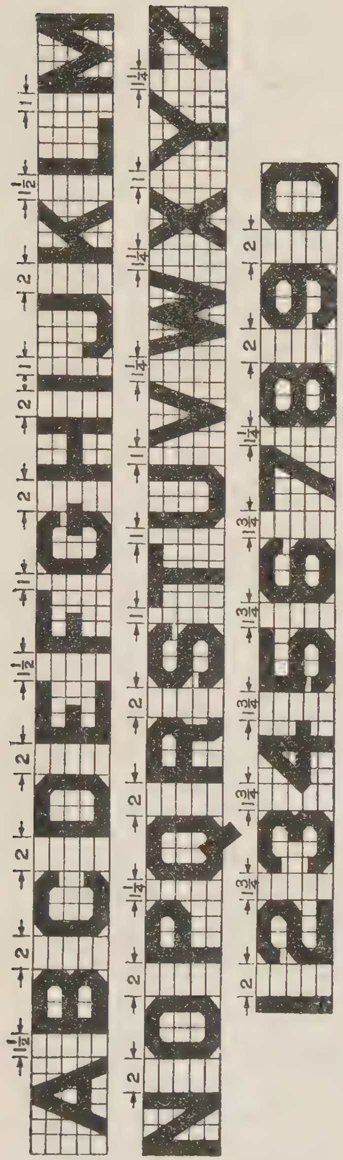


FIG. 9

height, and that of *W* is 1.1 times the height. The total widths of the letter *G* and figure *4* are each a little greater than four-fifths the height, as shown in the figure.

The widths of the spaces between the letters depend on the arrangement of the letters in the words, and are different for different arrangements. The letters in Fig. 9 were spaced to present the best possible appearance for that arrangement. The spaces in the figure are given in parts of the height, the unit being one-fifth the height. For example, the space between *B* and *C* is given as *2*, indicating that this space is 2 parts, or two-fifths of the height. It is difficult to give rules for the spacing of letters in every case that may arise; the best spacing is done by eye so as to give a smooth unbroken appearance. If the letters are too close together, they appear crowded and are difficult to read; if they are too far apart, the open spaces between them give the drawing a broken appearance. Where the letter *A* is adjacent to *F*, *P*, *T*, *V*, *W*, or *Y*, the

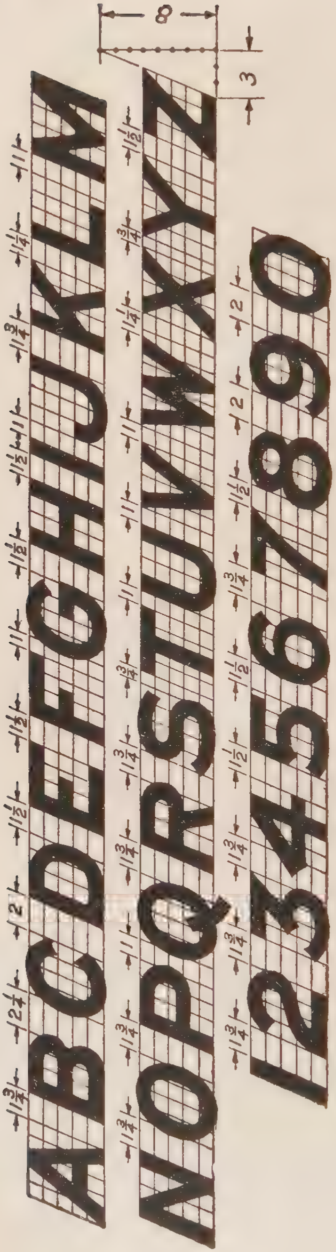


Fig. 10

lower extremity of *A* is placed vertically under the extremity of the other letter. The same rule applies to the letters *F*, *J*, *L*, *P*, *T*, *V*, *W*, and *Y*, whenever any of them is adjacent to another letter in such a way as to make this spacing possible. On the Practice Sheet, presently to be described, the student will lay out the letters according to the guide lines shown in Fig. 9.

46. Inclined Block Letters.—A better form of lettering for freehand work is shown in Fig. 10. The general proportions and spacing are the same as explained for the vertical letters in the preceding article. Many of the letters shown in Fig. 10 cannot be made by means of the ruling pen; hence, all should be made freehand, as a more uniform result will thus be obtained. If the letters having curved outlines are made exactly the same height as those having straight outlines, they will appear smaller; for this reason, round letters, such as *C*, *G*, *O*, are made slightly higher than the others. The inclined guide lines for these

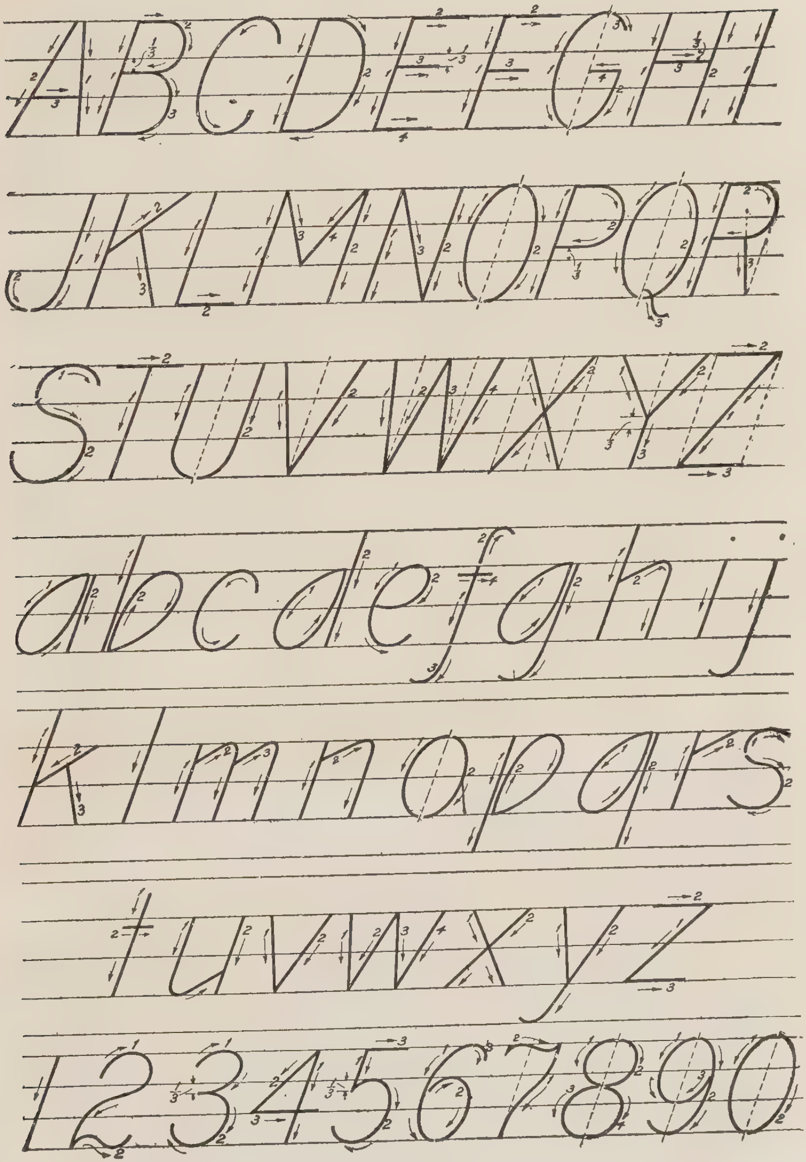


FIG. 11

letters make an angle with the horizontal guide lines of eight vertical to three horizontal, as shown at the right end of Fig. 10, and it is a great help to rule them in, in light pencil lines, so as to keep the slope of the lettering uniform. The lines may be made almost any thickness up to nearly one-fourth the height, but clearer and more legible words are obtained if they are made not greater than one-fifth the height.

47. Working Letters.—On all drawings showing constructive features, there are numerous notes and descriptive matter. The neatness of the drawing depends to a great extent on the lettering of these notes, both as to style and execution. The style that has become general on working drawings, on account both of the rapidity with which it may be executed and of its clearness and legibility, is known as the *single stroke*, and is shown in Fig. 11. This figure shows the number of pen strokes in each letter as well as the

Riveted Joints Riveted Joints Riveted Joints

FIG. 12

direction and the order in which the strokes should be made. The student should observe carefully the formation of these letters and follow the order and direction of the strokes given in the figure. Each stroke is numbered, and the direction is marked by an arrow. The student is advised to practice these letters assiduously, making them at least the size shown in the figure, as, by making them large, imperfections will show more plainly and he can better see where improvement is needed.

By comparing this style of lettering with the single-line Italic employed for similar descriptive matter on the plates in *Geometrical Drawing*, it will be noticed how much simpler is the style shown in Fig. 11. The single-stroke letter has the further advantage of occupying less space lengthwise than any other style of letter, and, even though the letters are crowded, the legibility is not impaired, as will be observed in Fig. 12.

The width and spacing of these letters may be made in one of the ways shown in Fig. 12, according to the amount of room available.

The alphabet shown in Fig. 11 is of value to the draftsman and engineer, and the student should by continual

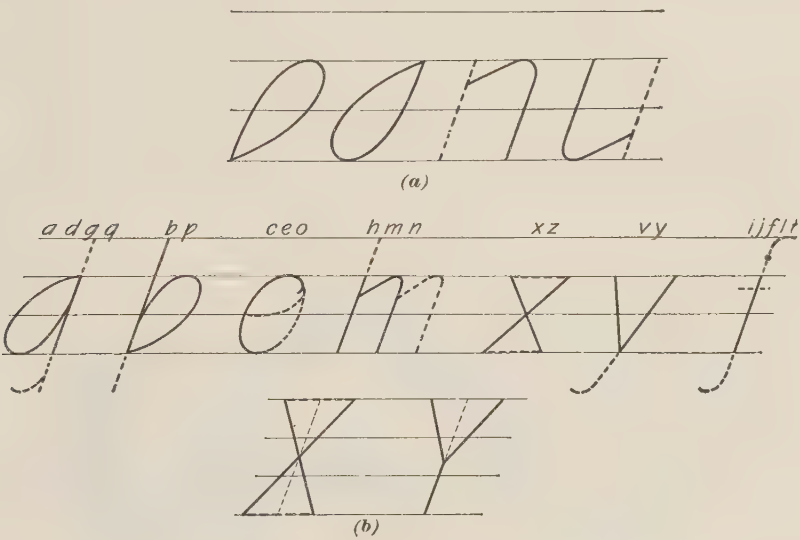


FIG. 13

practice familiarize himself with the formation and proportion of these letters. The elements of the letters are formed by a single pen stroke, and, when executed at a uniform angle and properly spaced, they present a line of great neatness and legibility. The principles on which these letters are constructed are shown in the ovals of Fig. 13 (*a*), as are also the characteristic curves by which such letters as *m*, *n*, and *u* are joined at the top or bottom. At (*b*), the similarity of certain letters is shown, which if carefully noted will assist in obtaining uniformity in lettering. Although a slope of 60° may be used for these letters, the

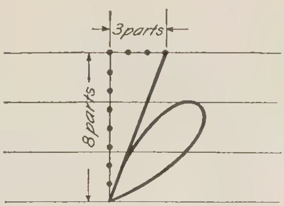


FIG. 14

best slant is that of the hypotenuse of a triangle having a base of three parts to a height of eight, as shown in Fig. 14.

In executing this lettering, the principal points to be observed are to keep the letters the same height and slant and as regularly spaced as possible. It is better, where neat lettering is required, to draw four or five guide lines for the letters, as shown in Fig. 11. The four spaces in the figure are equal; the top line limits the height of the capitals and of such letters as *b*, *d*, *f*, *h*, *k*, *l*, etc.; the second line from the top locates the height of the small letters. Such letters as *g*, *j*, *p*, *q*, and *y* extend beyond the fourth line a distance

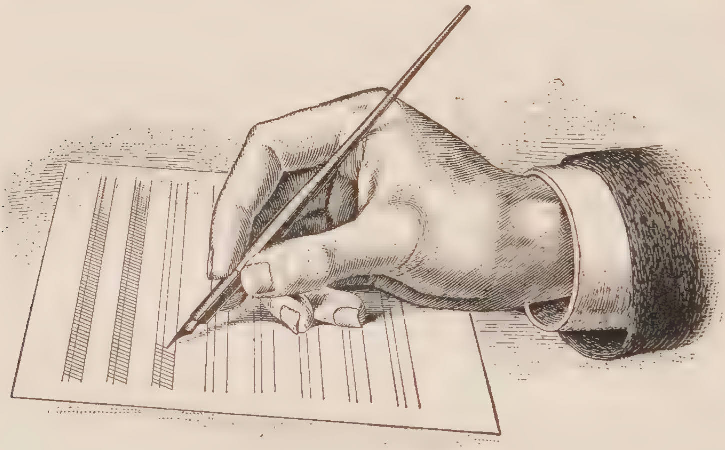
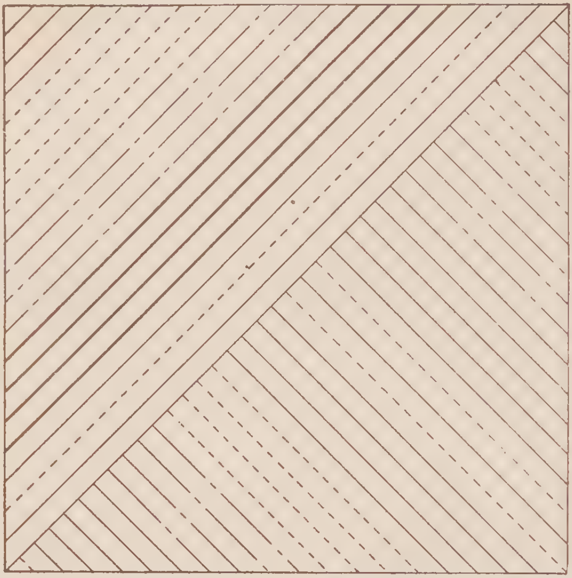


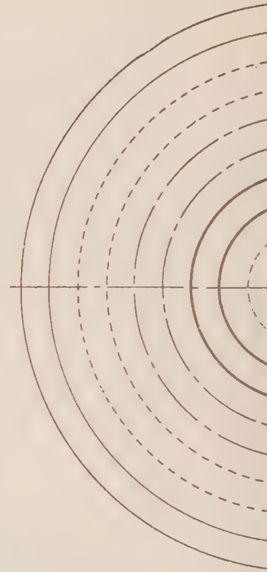
FIG. 15

equal to one space. It is, however, unnecessary to draw a line for this purpose, as these letters are usually at such a distance from one another that a slight deviation is indiscernible and the eye is a sufficient guide. A good exercise for the student who wishes to become proficient in the execution of this type of lettering is shown in Fig. 15; it consists in drawing, freehand, numerous lines about the height of the large letters, all parallel and at the proper angle with the horizontal.

There are many devices that the ingenious draftsman may adopt in order to facilitate the lettering on a drawing.

Fig. 1.





LETTERS

TYPE I. ABCDEFGHIJKLM

NOPQRSTUVWXYZ

1234567890

TYPE II. *ABCDEFGHIJKLM*

NOPQRSTUVWXYZ

1234567890

LINES

Fig. 5.

SURFACE & CUTTING-PLANE LINE

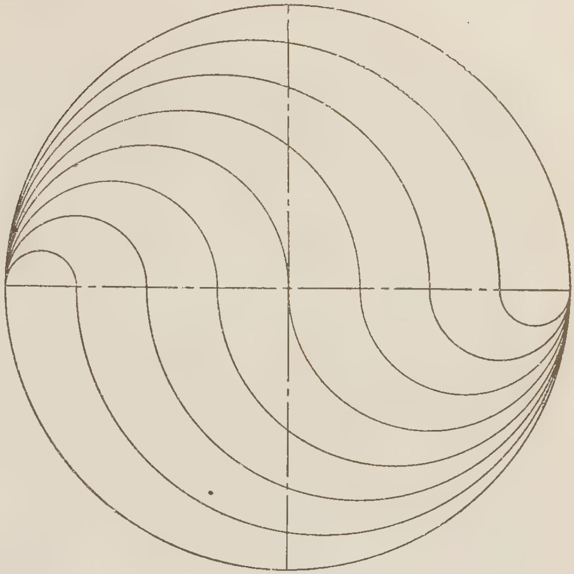
HIDDEN SURFACE LINE

DIMENSION

CENTER LI

2.

Fig. 3.



FIGURES

TYPE III —— A B C D E F G H I J K L M N O P Q R S T U V W X Y Z

1 2 3 4 5 6 7 8 9 0

DECIMALS 0.025 0.075 .1095 FRACTIONS $\frac{1}{8}$ $\frac{9}{16}$ $\frac{33}{67}$

TYPE IV —— a b c d e f g h i j k l m n o p q r s t u v w x y z &c.

NOTE: —— THE OVAL IN TYPE IV IS SHAPED THUS O AND WHEN INVERTED THUS O. THIS OVAL OR PART OF IT IS USED IN THE FOLLOWING LETTERS IN MANNER INDICATED.

a b c d e g h j m n p q r

PRACTICE SHEET

DRAWN BY
DATE

101

It is a common practice, where there is much descriptive matter that has not been written on the pencil drawing but must be written on the tracing, to place under the tracing cloth a sheet of ruled cross-section paper and use the rulings as guide lines for the lettering.

48. Neatness.—It should be constantly borne in mind that the work of a draftsman should be accurate and neat; that all lines, figures (numbers), and letters must be clear cut and distinct; that there must be no doubt as to the meaning of lines and dimensions; that mistakes made on drawings are often more serious than errors in the field or shop, for they may not be located until the various parts of the work are put together. Even though the tracer realizes, in practice, that all his work will be checked, he should not relax his vigilance on that account. If a man is inaccurate, the checker soon finds it out, and he is usually not slow in imparting the information to the chief draftsman.

49. Sending the Work.—The student will send to the Schools tracings of all the remaining plates of this Course. He should not send the pencil drawings, which he should retain, as it may be necessary for him to retrace them.

PLATE 101, TITLE: PRACTICE SHEET

50. Introduction.—This plate is intended for a practice sheet in tracing, and is introduced to give the student an opportunity to familiarize himself with the methods of making lines and letters on tracing cloth; it is not in any way a working drawing.

In order to distinguish readily between references to figure numbers on the drawing plates and those in the text, the former will be designated by a star. Thus, Fig.* 1 indicates figure 1 on the drawing plate under consideration. The usual abbreviation Fig. refers to figures in the text.

51. Directions for Laying Out the Pencil Drawing. Fig.* 1 is a set of lines of different kinds and weights inclined

at angles of 45° to the horizontal, and enclosed in a square 5 inches on a side. Locate the top line of the square $\frac{3}{4}$ inch from the top border line, and the left side $\frac{1}{2}$ inch from the left border line. Divide each of the four sides of the square into 19 equal parts in one of the ways described in *Geometrical Drawing*. Draw the lines from the points of division with the aid of the 45° triangle resting on the T square. In the pencil drawing, the lines are all drawn of the same weight, and, if desired, all full. In tracing them, the pen should be adjusted to obtain the proper widths. At first, it is better for the student not to draw full those lines that are to be composed of dots or dashes, but to draw them in pencil just as they are to appear on the tracing, with the exception of the weight or width.

52. Fig.* 2 is a series of ten concentric circles, the outer circle being 5 inches in diameter. First draw the two diameters at right angles to each other, locating the vertical diameter at the center of the plate and the horizontal diameter $3\frac{1}{4}$ inches from the top border line. The radii of the circles can be laid off along the horizontal diameter by means of the full-size scale. They are $\frac{1}{4}$, $\frac{1}{2}$, $\frac{3}{4}$, 1, $1\frac{1}{4}$, $1\frac{1}{2}$, $1\frac{3}{4}$, 2, $2\frac{1}{4}$, and $2\frac{1}{2}$ inches, respectively. Do not bear on the point of the compass at the center very hard, or a bad hole will be worn in the tracing. As in the case of Fig.* 1, the pencil lines may all be made the same weight if desired.

53. Fig.* 3 consists of several semicircles enclosed in a circle 5 inches in diameter. First draw the two diameters at right angles to each other, placing the vertical diameter 3 inches from the right border line and the horizontal diameter $3\frac{1}{4}$ inches from the top border line. Draw the outside circle 5 inches in diameter. Mark off the horizontal diameter into 16 equal parts of $\frac{5}{16}$ inch each by means of the full-size scale. Use each of the points of subdivision as the center of a semicircle, and draw the semicircles as shown. The radii of the semicircles are, respectively, $\frac{5}{16}$, $\frac{5}{8}$, $\frac{15}{16}$, $1\frac{1}{4}$, $1\frac{9}{16}$, $1\frac{7}{8}$, and $2\frac{3}{16}$ inches. Great care must be taken in laying off the distances along the horizontal diameter and in drawing

the semicircles, that they will be exactly tangent, as shown, and meet the outside circle at the ends of the diameter. The object of this exercise is to teach the student to connect the curves by smooth lines without any breaks or irregularities.

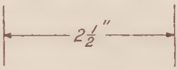
54. Fig.* 4 represents various kinds of lettering that will be used in this and succeeding plates. They are made as described in previous articles, and no further instructions are necessary, except as to their height. The letters in the heading *Letters & Figures* are $\frac{1}{4}$ inch in height; their thickness is about one-sixth the height, but the student may make the thickness equal to one-fifth the height, as in Fig. 10, if he so desires. The letters and numerals in Type I and Type II are all $\frac{1}{2}$ inch in height, and are the same as the alphabets shown in Figs. 9 and 10, respectively. The letters and numerals in Type III are $\frac{1}{8}$ inch high, and the fractions are $\frac{1}{4}$ inch high; the letters and numerals are made as shown in Fig. 11. In Type IV, the stems of such letters as *b*, *d*, and *h* are $\frac{1}{8}$ inch high, and the main parts of the letters are made two-thirds this height. In the note, the letters are $\frac{3}{32}$ inch in height, and in the last line of letters the stems are $\frac{1}{4}$ inch in height. The student is advised to draw the letters of each type on small sheets of paper several times, so as to become familiar with the method of forming them before putting them on the plate. This will give him a good idea of the amount of room to leave for each letter, so that he can arrange the different types laterally about as shown on the plate. The lower guide line of the heading is $6\frac{9}{16}$ inches above the lower border line; the lower lines of Type I are, respectively, $5\frac{3}{4}$, 5, and $4\frac{1}{4}$ inches above the lower border line; of Type II, $3\frac{1}{8}$, $2\frac{3}{8}$, and $1\frac{5}{8}$ inches; of Type III, 6, $5\frac{5}{8}$, and $5\frac{3}{16}$ inches; of Type IV, $4\frac{5}{16}$ inches; of the note, $3\frac{7}{16}$, $3\frac{3}{16}$, and $2\frac{15}{16}$ inches, and of the last line of letters, $2\frac{3}{8}$ inches. In drawing the letters on the plate, it will be best for the student to draw all necessary horizontal, inclined and vertical guide lines. He can then follow the instructions given in preceding articles in forming the letters. In practice, it is not customary to draw all the guide lines, as an experienced

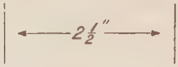
draftsman can usually form the letters by eye, with the aid of the lower guide line alone.

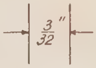
55. Fig.* 5 shows the different kinds of lines that are commonly used in mechanical and construction drawing. In drawing them on the tracing, they should in every case be made slightly heavier than shown on the plate, since the plate is somewhat reduced in size. The letters in the heading *Lines* are $\frac{1}{4}$ inch high, the same as for the heading for Fig.* 4, and they are made the same as Type II, Fig.* 4. The lower guide line of the heading is 1 inch from the lower border line. The names of the lines are $\frac{1}{8}$ inch high, the letters being the same as those of Type III, Fig,* 4. The lower guide lines for those letters are, respectively, $\frac{3}{16}$ and $\frac{1}{2}$ inch from the lower border line. Each line, including the heading, is $5\frac{1}{2}$ inches in length.

DEFINITIONS OF LINES

56. Surface lines are lines that limit or bound the delineation of an object, and should be of the same weight all through the figure.

57. Dimension lines are lines drawn to show limits to which dimensions apply, and are ended with an arrowhead, put in freehand; the arrowheads should always touch the lines that limit the dimension, thus: 

; never thus: 

. A short dimension may be indicated thus: 

, placing arrowheads outside of the limiting lines, or thus: 

Do not put dimension lines in or over the delineation of parts, nor place figures where they cannot be easily read.

58. Center Lines.—When objects are symmetrical, or nearly so, **center lines** are usually drawn at the working

center of the drawing. These lines are very important, being lines of reference from which the work is to be laid out. Dimensions are frequently given from the center lines, which always serve as a guide for all dimensions, even when not used to limit the dimensions. In making working drawings, it is important that one of the center lines should be first located, as this will serve as a general line of reference. This center line is not necessarily the geometrical center of the figure; it may be simply an important line in the object drawn, from which or along which, as a basis, dimensions are laid off.

This line is also used to indicate where a section has been or is to be taken. If, however, a partial section is indicated, the remainder of the view being in elevation or plan, the full (surface) line is used, except where there is an opening appearing in both parts of the view; in this case, the dash and dot would be used where the line crosses the opening, and the full line where it crosses the solid portion.

59. **Hidden surface lines** are lines drawn to show the surfaces of hidden parts. It should be noticed that a part hidden in one view is sometimes drawn full in other views, indicating that from one position the part cannot be seen, while from the other position it can.

60. As previously stated, all lines on tracings should be fully 50 per cent. heavier than the lines used on the plates drawn in connection with *Geometrical Drawing*. The only case in which a fine line should be used is when locating points by intersecting lines, and even then they are not actually needed, since the points can be located by fine pencil lines, and being once located the lines passing through the points may be as heavy as desired. Fine lines on tracings render it very difficult to obtain a good blueprint.

61. It will be noticed that the broken-and-dotted lines, here used for center lines and to indicate where a section is to be, or has been taken, consist of a repetition of long dashes, each followed by one dot. In *Geometrical Drawing*, two dots were used. Both forms of this line are widely

employed, but the student is advised to use the one here shown, as it is simpler. When working in a drafting room, however, a draftsman must conform to the standards in force there.

PLATE 102, TITLE: MASONRY AND TIMBER

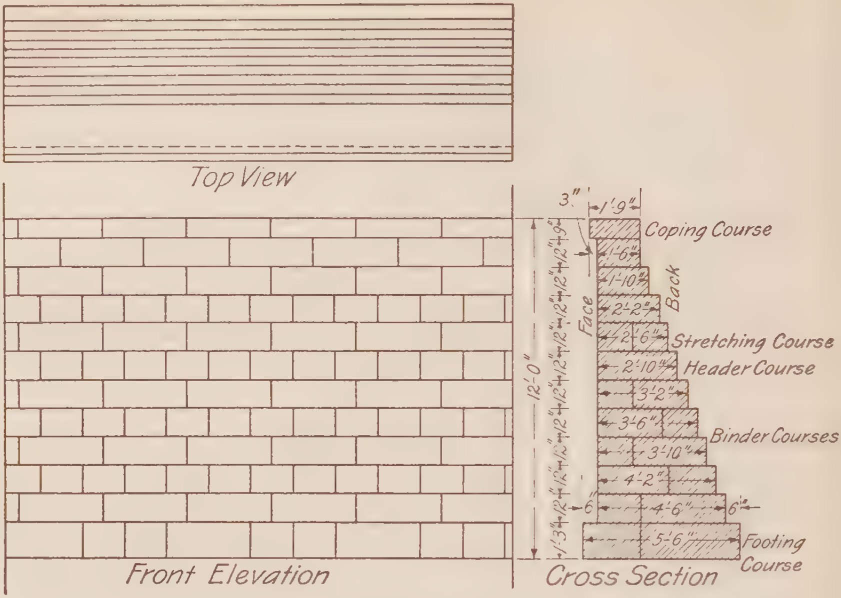
62. Introduction.—This plate is intended as a practice sheet to teach the methods employed to represent different kinds of masonry, as well as masonry and timber construction.

63. Masonry Retaining Wall.—Fig.* 1 shows the front elevation, top view, and cross-section of a retaining wall of stone masonry. The principal dimensions are given, together with the names usually applied to the various parts of the wall. The vertical height of each course is given, no allowance being made for joints. This is quite common practice; the actual sizes of the stones in the different courses depend on the kind of masonry, and are usually computed by the stone cutter from the dimensions given on the drawing. In the front elevation, the longer stones (**stretchers**) are shown 3 feet long, and the narrower stones (**headers**) are shown 18 inches in width.

The three views are located on the plate as follows: The top line of the front elevation is $2\frac{1}{4}$ inches from the top border line, the left vertical cutting line is $\frac{3}{4}$ inch from the left border line, and the front elevation is $4\frac{1}{2}$ inches long. The front line of the top view is $\frac{1}{2}$ inch from the top line of the front elevation. The face of the abutment, as shown in cross-section, is **plumb**, that is, vertical, and $\frac{3}{4}$ inch from the right vertical cutting line of the front elevation.

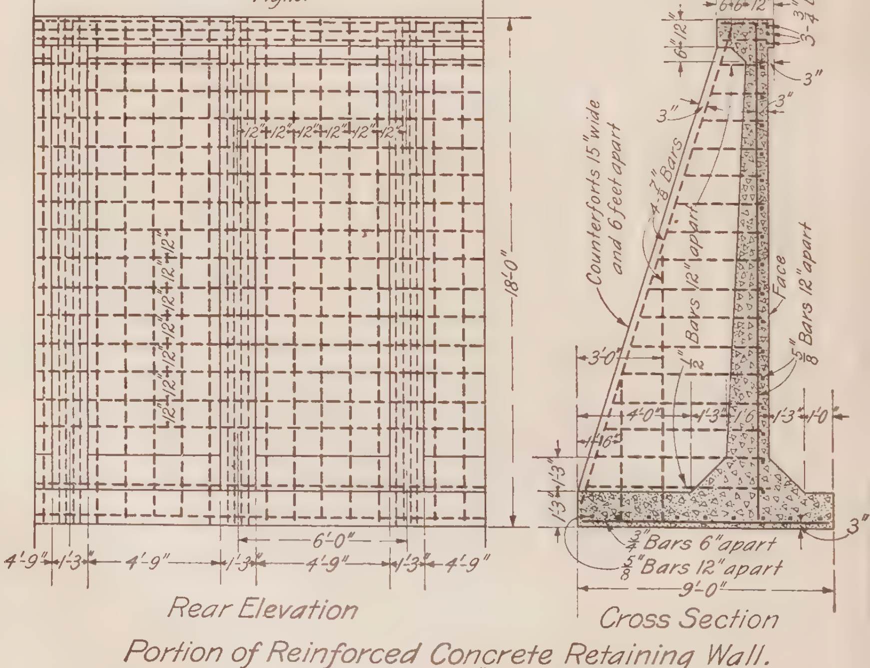
With these lines laid out on the plate, the dimensions are then laid out to the scale of $\frac{1}{4}$ inch to the foot; that is, each foot on the wall is shown as $\frac{1}{4}$ inch on the drawing. For example, the intermediate courses or layers of stone measure each $\frac{1}{4}$ inch on the drawing, and are 1 foot thick in the actual wall. The best results will be obtained if the cross-section is laid out to scale before the remainder of the figure is drawn. The other views can then be projected as shown.

Fig. 1.



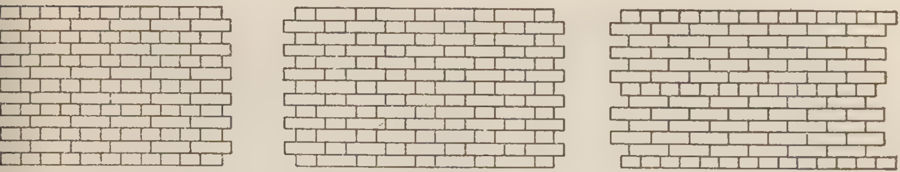
Portion of Stone Retaining Wall.
Scale $\frac{1}{4}$" = 1'.

Fig. 2.



Portion of Reinforced Concrete Retaining Wall.
Scale $\frac{1}{4}$" = 1'.

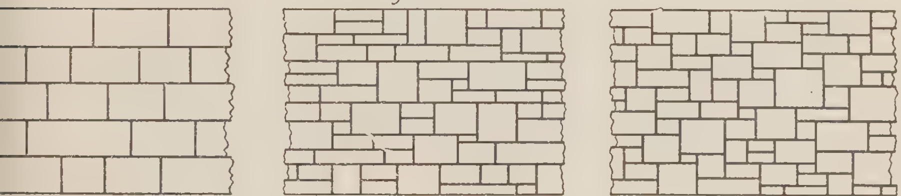
Fig. 3.



English Bond — Flemish Bond — Common Bond

Brick Masonry.

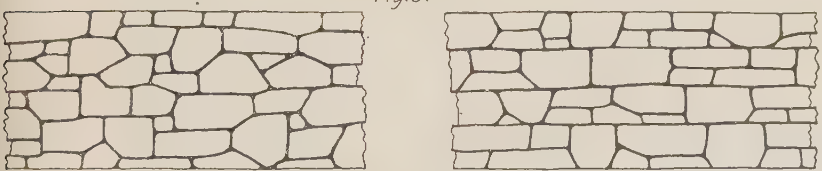
Fig. 4.



Range — Broken Range — Random Range

Squared Stone and Ashlar Masonry.

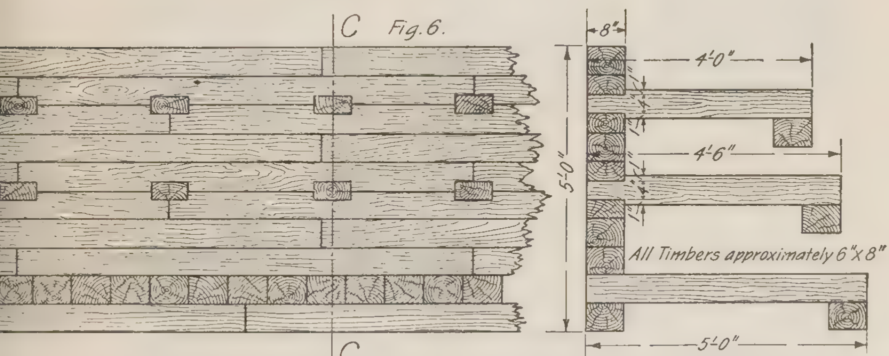
Fig. 5.



Uncoursed — Coursed

Rubble Masonry.

Fig. 6.



Front Elevation. — Cross Section on C-C

Timber Retaining Wall.

Scale $\frac{1}{2}''=1'$.

MASONRY AND TIMBER

DRAWN BY ____________

DATE ____________

102

In laying out these dimensions to scale, the general instructions for scaling given in Arts. 28 and 29 must be followed. Only the outlines, joints, and dimensions need be given on the pencil drawing, the fine parallel lines shown in the interior of the cross-section being omitted on the pencil drawing and drawn on the tracing. These lines are called **section lines,** or **cross-hatching,** and are the same distance apart all over the section. There are several special appliances for ruling section lines, but in the plates of this Course all section lines will be drawn without the use of a scale or special instrument, the eye being the only guide in spacing them evenly, and the student will follow this plan in drawing the sections shown. With practice, great skill can be acquired in spacing the lines evenly, and the work can be done very rapidly.

64. Reinforced-Concrete Retaining Wall.—Fig.* 2 shows the rear elevation and cross-section of a retaining wall of reinforced concrete. The principal dimensions of the wall, together with the size and spacing of the reinforcing rods, are given in the figure. The dotted lines in the rear elevation represent the reinforcing rods; the full vertical

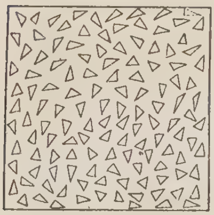
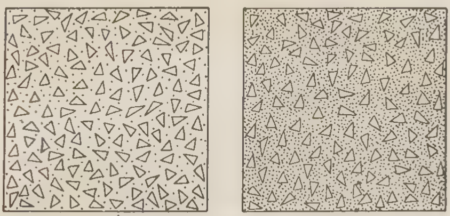


Fig. 16



(a) (b)

Fig. 17

lines are the outside surfaces of the counterforts. In the cross-section, the reinforcing rods in the counterfort are shown by dotted lines; in this view, the rods that are cut by the section are shown as small black squares, and those parallel to the section as heavy black lines; this is the usual practice for this type of construction. This view also illustrates the usual method of representing broken-stone concrete. In forming the concrete, it is customary first to draw

at random throughout the section a number of irregular triangles, as shown in Fig. 16; these are assumed to represent the irregular broken stones. When a sufficient number have been drawn, the remaining area, between the stones, is filled with small dots. The section can be shown light by putting in only a few dots, as in Fig. 17 (*a*), and dark by showing a large number of dots, as in Fig. 17 (*b*). The student should try to get approximately the same shade as that used in the plate. For this kind of work, a Gillott's 303 or some other kind of fine pen should be used.

65. When gravel is used in the concrete instead of broken stone, it is customary first to draw a number of

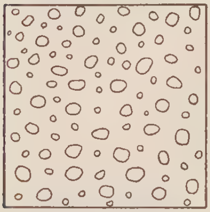
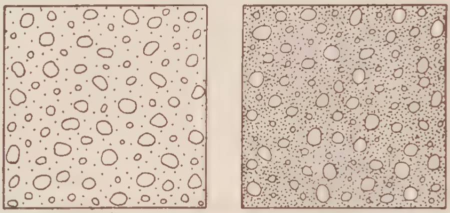


FIG. 18



(*a*) FIG. 19 (*b*)

irregular rings or closed figures, as shown in Fig. 18. The remaining area is then filled in with small dots the same as for broken-stone concrete (see Fig. 19).

66. The two views in Fig.* 2 are located on the plate as follows: The bottom line of the rear elevation is $1\frac{1}{2}$ inches above the lower border line; the left vertical cutting line is $\frac{3}{4}$ inch from the left border line, and the rear elevation is 4 inches wide. The face of the retaining wall, shown in the cross-section, is plumb, and is located $7\frac{1}{4}$ inches from the left border line. With these lines located on the plate, the remaining lines are laid out, according to the dimensions given, to a scale of $\frac{1}{4}$ inch to the foot. It is the best plan to lay out the entire cross-section before drawing the rear elevation; then the lines can be projected across from the section to the elevation.

67. Brick Masonry.—Fig.* 3 shows the front elevations of portions of brick walls laid in different styles.

The first portion on the left is called **English bond.** In this style, the courses are alternately stretchers (that is, bricks with the long side at the surface of the wall) and headers (that is, bricks with the short side at the surface of the wall).

The second portion is called **Flemish bond.** In this style, the bricks in every course are alternately headers and stretchers.

The third portion is called **common bond.** In this style, five to ten courses are laid with stretchers, and then one course with headers.

The standard form of brick measures about $8\frac{1}{4}$ in. $\times$ 4 in. $\times$ $2\frac{1}{4}$ in., and the joints are, as a rule, $\frac{1}{4}$ inch thick. The lines representing the joints allow for the thicknesses of the joints, and are, respectively, $2\frac{1}{2}$, $4\frac{1}{4}$, and $8\frac{1}{2}$ inches apart. They are laid out to a scale of $\frac{1}{2}$ inch to the foot, the top line of each being 1 inch from the top border line. The student can arrange the three parts of the figure on this line about as shown in the plate.

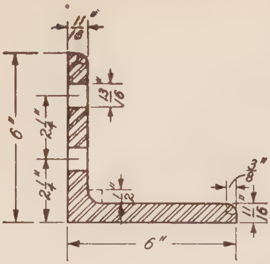
68. Squared-Stone and Ashlar Masonry.—Fig.* 4 shows the front elevation of three kinds of squared-stone masonry. In **range masonry,** all the horizontal joints are continuous; in **broken range,** the horizontal joints are partly continuous; in **random range,** there is no continuity in the horizontal joints. The top lines of these figures are $3\frac{1}{2}$ inches from the top border line; the figures are $1\frac{5}{8}$ inches high. The student can arrange the joints and also the figures on the sheet approximately as shown on the plate. There is no need, in the case of the broken-range and random-range masonry, for the student to arrange the joints exactly as shown on the plate. It will be better practice for him to exercise his ingenuity and arrange them to suit himself.

69. Rubble Masonry.—Fig.* 5 shows two kinds of rubble masonry. In the **uncoursed** rubble masonry, no joints, either vertical or horizontal, are continuous; in the **coursed** rubble masonry, the stone is finished off horizontal at occasional intervals. The top line of each figure is

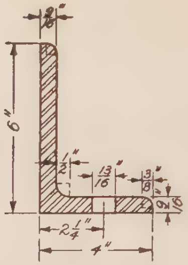
$6\frac{1}{8}$ inches from the top border line, and each figure is $1\frac{3}{8}$ inches high. The student should follow the same general arrangement as shown on the plate when he arranges the joints, but there is no need to copy or reproduce them exactly. To get the best results, these joints should be drawn in freehand.

70. Timber Retaining Wall.—Fig.* 6 shows the front elevation and cross-section of a timber retaining wall, and illustrates the usual methods of representing the end and side views of timber. The top line of this wall is $4\frac{1}{2}$ inches, and the bottom line 2 inches above the lower border line. The front line of the cross-section is plumb, and is 3 inches from the right border line. This style of wall is frequently used for temporary purposes, and when so used is most often composed of second-hand railroad ties or other second-hand material. It should not be considered in connection with permanent work. The timbers are approximately 6 in. × 8 in. × 8 ft., framed to 4 inches thick where the transverse pieces enter the face of the wall. The dimensions given on the plate are laid off to a scale of $\frac{1}{2}$ inch to the foot.

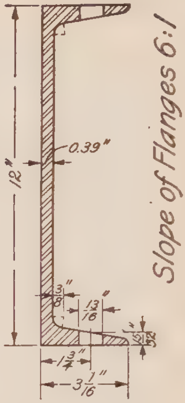
71. Concluding Remarks.—It is a waste of time to put cross-section lines, conventional signs for concrete, the wood marks on timber, etc. on pencil drawings, as better results can usually be obtained if these are made directly on the tracings. All dimension lines, dimensions, notes, etc. should be put on the pencil drawing to assist in locating them properly when tracing. It is also well in the beginning to draw guide lines for all dimensions and letters. The heights of letters on this and succeeding sheets were chosen so as to give the letters the proper appearance with relation to the remainder of the drawing. The student should try to make his letters about the same height as those shown on the plate, but there is no necessity for him to make them exactly the same height.



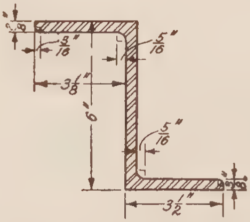
6″x 6″x $\frac{11}{16}$″ L
Fig.1.



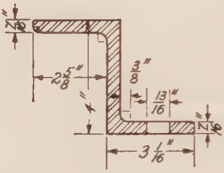
6″x 4″x $\frac{9}{16}$″ L
Fig.2.



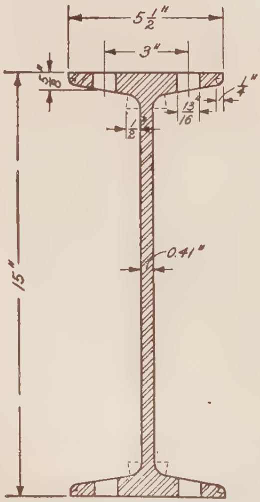
12″-20.5 #-[
Fig.3.



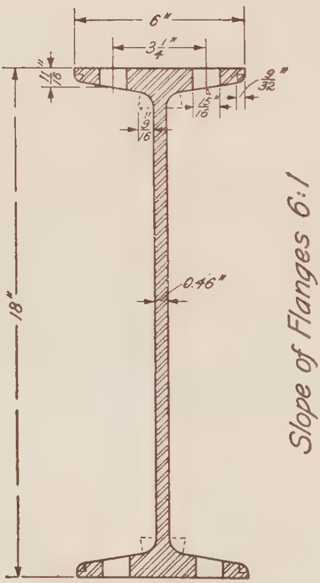
$3\frac{1}{2}$″x 6″x $3\frac{1}{2}$″x $\frac{3}{8}$″ Z
Fig.6.



$3\frac{1}{16}$″x 4″x $3\frac{1}{16}$″x $\frac{7}{16}$″ Z
Fig.7.

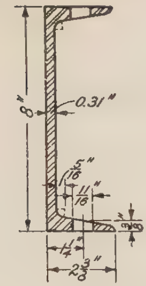


15″-42 # I-Beam.
Fig.8.

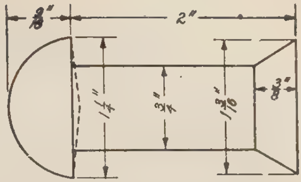


18″-55 # I-Beam.
Fig.9.

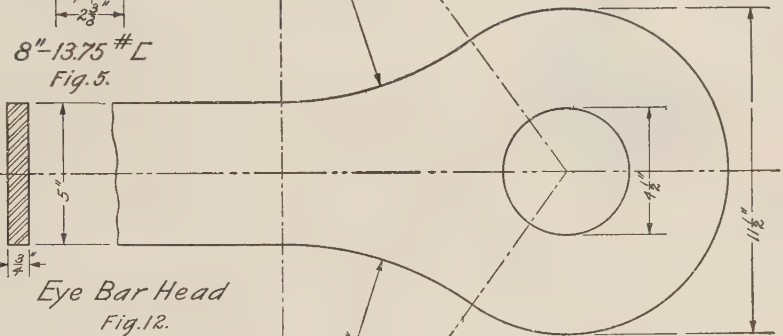
24″-80 #
Fig



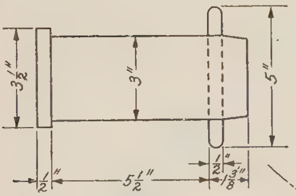
8"-13.75 #C
Fig. 5.



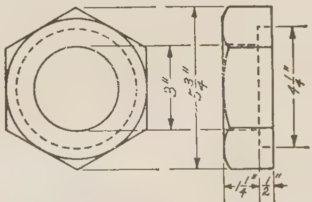
3/4" Countersunk Rivet
Fig. 11.



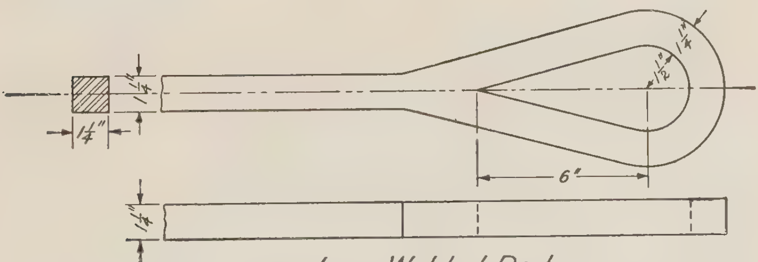
Eye Bar Head
Fig. 12.



Lateral Pin with Cotter
Fig. 13.



Pin Nut
Fig. 14.



Loop Welded Rod
Fig. 15.

STRUCTURAL
STANDARDS

DRAWN BY
DATE

103

PLATE 103, TITLE: STRUCTURAL STANDARDS

72. Introduction.—This plate shows some structural standards much used in civil-engineering work. Figs.* 1 to 10 are cross-sections of standard structural shapes; Figs.* 11 to 15 show portions of other standards.

73. Angles.—The shapes shown in cross-section in Figs.* 1 and 2 are called **angles,** and are usually designated by the capital letter **L.** The two sides are at right angles and are called the **legs.** Fig. 20 shows a perspective view of an angle, together with the holes that are used for riveting the angle to another angle or some other piece of steel. The portions in Figs.* 1 and 2 with no cross-hatching represent open holes for rivets. The lower line of each angle is $2\frac{1}{4}$ inches below the top border line. The left line of Fig.* 1 is 1 inch, and that of Fig.* 2 is $3\frac{7}{8}$ inches from the left border line. The dimensions given on the plate are drawn to a scale of 3 inches to the foot.

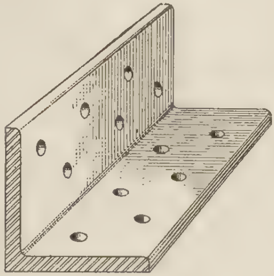


Fig. 20

74. Channels.—The shapes drawn in Figs.* 3, 4, and 5 are called **channels,** and are usually designated by the symbol **[.** Fig. 21 shows a perspective view of a channel, together with the holes that are left for rivets. The narrow vertical portion is the **web;** the remainder (the horizontal portions), are the **flanges.** The outer surfaces of the flanges are at right angles to the web. The inner surfaces of the flanges are not parallel to the outer surfaces, but slope, as shown on the plate, with a slope of 6 to 1. The top line of each channel is $\frac{5}{8}$ inch below the top border line; the left line of Fig.* 3 is $6\frac{3}{8}$ inches, that of Fig.* 4 is $8\frac{3}{8}$ inches, and that

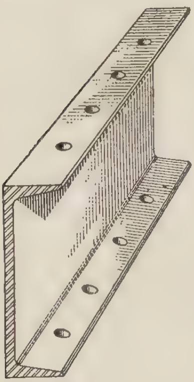


Fig. 21

of Fig.* 5 is $10\frac{1}{4}$ inches from the left border line. The lines given on the plate are drawn to a scale of 3 inches to the foot.

75. Z Bars.—The shapes shown in Figs.* 6 and 7 are called **zee bars**, and are usually designated by the symbol **Z**.

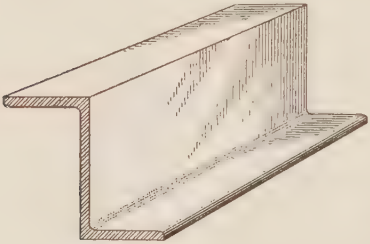


FIG. 22

Fig. 22 shows a perspective view of a **Z** bar. The vertical portion is the **web**; and the horizontal portions are the **flanges.** The flanges are at right angles to the web. In both Figs.* 6 and 7, the top line is 4 inches from the top border line; the left side of the web of Fig.* 6 is $1\frac{3}{4}$ inches, and that of Fig.* 7 is $4\frac{1}{2}$ inches from the left border line. The lines given are drawn to a scale of 3 inches to the foot.

76. I Beams.—The shapes shown in Figs.* 8, 9, and 10 are called **I beams,** from the similarity of the cross-section to the capital letter I. The vertical part is the **web;** the horizontal parts are the **flanges.** Fig. 23 shows a perspective view of an I beam, together with the holes in the top and bottom flanges. The outer surfaces of the flanges are at right angles to the web; the inner surface of the flanges are not parallel to the outer surfaces, but are inclined at a slope of 6 to 1, as shown on the plate. The lower line of each I beam is $1\frac{1}{4}$ inches from the lower border line; the center of the web in Fig.* 8 is $1\frac{3}{4}$ inches; in Fig.* 9, $4\frac{3}{4}$ inches, and in Fig.* 10, 8 inches, from the left border line. The shapes are drawn to a scale of 3 inches to the foot.

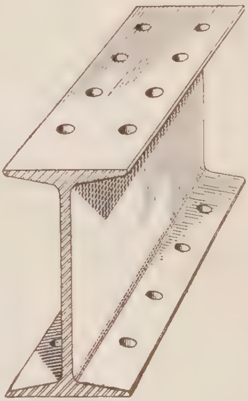


FIG. 23

77. Rivets.—Fig.* 11 shows the elevation of a steel rivet. The round head at the left end is called a **button**

head. The flat conical head at the right end is called a **countersunk head.** Most rivets have button heads at both ends, some at only one end, while a few have countersunk heads at both ends. The rivet given on the plate is drawn full size; the center of the curved outline of the bottom head is found by trial, so that the head will be $\frac{9}{16}$ inch in height and $1\frac{1}{4}$ inches in diameter. The center line of the rivet is $1\frac{1}{8}$ inches from the top border line, and the right end is $\frac{1}{2}$ inch from the right border line.

78. Eyebar Head.—Fig.* 12 shows an eyebar head. The bar is rectangular in cross-section through the greater part of its length, and is increased in width at the end so as to form a circular head. A hole is bored in the center of this head to allow the connection of the eyebar to other pieces. The center line of the bar is $4\frac{3}{8}$ inches from the top border line; the center of the circular head is $2\frac{1}{4}$ inches from the right border line. The left end of the eyebar is shown broken off by an irregular line about $6\frac{1}{4}$ inches from the right border line. The right side of the cross-section is 7 inches from the right border line. The figure is drawn to a scale of 3 inches to the foot.

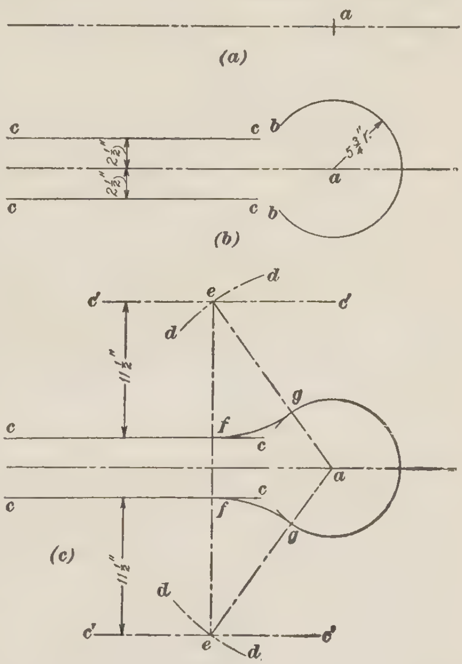


Fig. 24

The centers of the circular curves that connect the straight sides of the bar with the outline of the circular head can be

found as follows: First draw the center line of the bar, Fig. 24 (a), and mark on it the location of the center a of the circular head. With a radius of $11\frac{1}{2} \div 2 = 5\frac{3}{4}$ inches, to a scale of 3 inches to the foot, and a as a center, draw the portion $b\,b$ of the outline of the circular head, as shown in Fig. 24 (b). Lay off the width of the bar, one-half on each side of the center line, and draw the lines $c\,c$ parallel to the center line representing the edges of the bar. Lay off, above and below the lines $c\,c$, distances of $11\frac{1}{2}$ inches, and draw the construction lines $c'\,c'$, Fig. 24 (c), parallel to $c\,c$. With a as a center, and a radius equal to $11\frac{1}{2} + 5\frac{3}{4} = 17\frac{1}{4}$ inches, describe the circular arcs $d\,d$, intersecting the lines $c'\,c'$ at e. Then the points marked e are the centers of the curves. Draw the line $e\,e$; this line intersects $c\,c$ at f, the beginning of the curve. Draw the lines $e\,a$; these lines intersect the circle $b\,b$ at g, g, the points where the curves join. With the points e as centers and a radius of $11\frac{1}{2}$ inches, the arcs $f\,g$ can now be drawn.

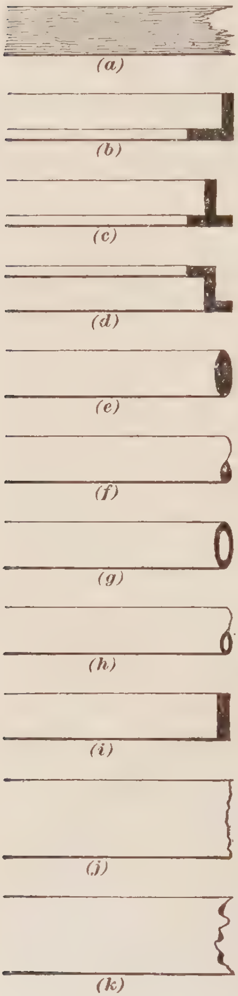


Fig. 25

79. Breaks.—In Fig.* 12, the eye-bar is shown broken off at the left end. This is frequently done when it is not desired to show the whole of a part. In many cases, the end is so broken or finished as to indicate the approximate shape of the object broken. Conventional methods of indicating breaks are shown in Fig. 25. Wood is usually shown broken in the manner illustrated at (a), angles as at (b), **T**'s as at (c), **Z** bars as at (d). Cylindrical objects are occasionally

broken as shown at (*e*), but most frequently in the manner shown at (*f*). Pipes and similar hollow cylindrical objects may be broken as shown at (*g*); but more frequently the break is made as shown at (*h*). Rectangular objects may be broken in the manner shown at (*i*). Plates and objects other than those included between views (*a*) and (*i*) are often shown broken off by drawing a wavy freehand line, as in (*j*) and (*k*). These methods will be followed in the subsequent plates of this Course.

80. Lateral Pin With Cotter.—Fig.* 13 is a side view of a **lateral pin** with a **cotter** in the end. This pin is circular in cross-section and somewhat resembles a bolt with a large diameter. The diameter at the right end is about $\frac{1}{4}$ inch less than the body of the pin—that is, about $2\frac{3}{4}$ inches. The center line of the pin is $6\frac{3}{4}$ inches below the top border line; the right end is $5\frac{3}{8}$ inches from the right border line. The figure is drawn to a scale of 3 inches to the foot.

81. Pin Nut.—Fig.* 14 is a top view and side view of a hexagonal pin nut. The center of the top view is $7\frac{1}{8}$ inches from the top border line, and $2\frac{3}{8}$ inches from the right border line. The right side of the side view is $\frac{5}{8}$ inch from the right border line. The figure is drawn to a scale of 3 inches to the foot. In drawing the hexagonal outline, the directions given in *Geometrical Drawing* for laying out a hexagon should be followed.

82. Loop-Welded Rod.—Fig.* 15 shows a cross-section and views of two sides of the end of a square **loop-welded rod.** The loop end is formed by bending the end of a straight rod into a circle and welding the end to the main part of the rod. No difficulty will be experienced in laying this out on the drawing. A scale of 3 inches to the foot should be used. The center line of the upper view is $3\frac{3}{8}$ inches above the bottom border line; the center of the lower view is $2\frac{1}{4}$ inches above the border line; the center of the cross-section is $6\frac{5}{8}$ inches, and the center of the circular portion of the end is $1\frac{3}{4}$ inches, from the right border line.

INDEX

NOTE.—All items in this index refer first to the section (see the Preface), and then to the page of the section. Thus, "Arch bricks, §41, p46," means that arch bricks will be found on page 46 of section 41.

G

H

M

N

O

Q

R

U

V

W